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Effect of Severe Corrosion on Lateral Strength of Square RC Bridge Columns

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ABSTRACT

Corrosion of reinforcing steel bars is the primary durability problem that causes degradation of reinforced concrete structures located in aggressive environments. Severe corrosion of steel bars decreases the lateral load-carrying capacity of reinforced concrete members, causes loss in the mechanical properties of reinforcement and cross-sectional area of steel bars and concrete cover, bond deterioration, reduces anchorage of steel bars, and decreases the confinement by transverse reinforcement. Consequently, corrosion results in drop in the lateral strength of columns. Therefore, studying response of corroded reinforced concrete columns subjected to lateral loads is necessary.

According to the Federal Highway Administration (FHWA) report in 2013, 25.9 percent of the total inventory of highway bridges are deficient or functionally obsolete. Corrosion damage caused by deicing salts is considered one of the main problems that cause a bridge structure to be structurally deficient (FHWA, 2004). However, absence of a practical model for assessment of the residual lateral strength of severely corroded RC columns as well as the lack of research on ultimate lateral capacity of deteriorated concrete structures shows the need to develop a practical method to calculate the current lateral capacity of corroded reinforced concrete bridge columns.

A new methodology was developed in this research, to evaluate the current lateral strength of severely corroded RC columns, which can be adapted to existing bridge condition evaluation methods. A Finite Element Analysis (FEA) model to simulate severely corroded columns was created and verified against experimental data conducted by other researchers. After being verified against experimental data, a total of 308 Finite element models were developed to investigate several variables that affect the lateral response of corroded columns. A series of $24 \text{ in} \times 24 \text{ in}$
square column sections having different material properties were modeled as cantilevers. Location of corrosion within the cross-section (Compression-side corroded, Tension-side corroded, All-sides corroded), corrosion level (CR=25%, 30%, 35%, 40%, 45%, 50%), length of corroded zone along the column height (1H=24in, 2H=48in), axial load ratio \( NR = \frac{P}{f'_{cA_g}} = 0\%, 5\%, 15\%, 25\% \), compressive strength of concrete \( f'_{c} = 4ksi, 7ksi \), steel reinforcing ratio \( \rho = 2\%, 3\%, 4\% \) and shear span to depth ratio \( L/d=2.5, 5 \) were the variables investigated in this study. For severely corroded RC columns, stirrups were assumed to be completely deteriorated and the concrete cover spalled off. Therefore, concrete cover and stirrups were removed at corroded locations. The corroded bars were assumed to be completely un-bonded to the surrounding concrete.

Based on results obtained from the finite element analysis, a practical model was developed. The proposed practical method considers all the changes in material and geometry properties including area loss of corroded steel bars and concrete cover, bond deterioration and its consequences on corroded bars’ buckling, location of corroded zone, length of corroded zone along the column, compressive strength of concrete, reinforcing ratio of RC column section, axial load ratio, and shear span to depth ratio. This study also provides engineers better understanding of lateral response of severely corroded RC bridge columns with detailed force-displacement diagrams based on finite element analysis.
Effect of Severe Corrosion on Lateral Strength of Square RC Bridge Columns

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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TABLE OF CONTENTS</td>
<td>VI</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>XII</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>XIX</td>
</tr>
<tr>
<td>1 INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Background</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Objectives and Scopes</td>
<td>2</td>
</tr>
<tr>
<td>1.3 Research Approach</td>
<td>3</td>
</tr>
<tr>
<td>1.4 Outline</td>
<td>3</td>
</tr>
<tr>
<td>2 LITERATURE REVIEW</td>
<td>5</td>
</tr>
<tr>
<td>2.1 Introduction</td>
<td>5</td>
</tr>
<tr>
<td>2.2 General Effects of Corrosion on RC Concrete Structures</td>
<td>6</td>
</tr>
<tr>
<td>2.2.1 Mechanism of Corrosion</td>
<td>7</td>
</tr>
<tr>
<td>2.2.1.1 Chloride-induced Corrosion</td>
<td>8</td>
</tr>
<tr>
<td>2.2.1.2 Carbonation-induced Corrosion</td>
<td>9</td>
</tr>
<tr>
<td>2.2.2 Corrosion Rate</td>
<td>11</td>
</tr>
<tr>
<td>2.2.3 Corrosion Effect on Mechanical Properties of Steel Bars</td>
<td>12</td>
</tr>
<tr>
<td>2.2.3.1 Bar Cross Sectional Area Loss due to Corrosion</td>
<td>12</td>
</tr>
</tbody>
</table>
2.2.3.2  Strength and Ductility Reduction due to Corrosion .............................. 14

2.2.3.3  Buckling of Compression bars due to Corrosion ................................. 16

2.2.4  Cracking of Concrete due to Corrosion ................................................. 17

2.2.5  Loss of Bond due to Corrosion .............................................................. 22

2.2.5.1  Bond Mechanism of Un-corroded RC Elements ............................... 23

2.2.5.2  Effect of Corrosion on Bond .............................................................. 27

2.3  Structural Behavior of Corroded Reinforced Concrete Elements ................. 30

2.4  Experimental Studies on Corroded RC members ..................................... 37

2.4.1  Studies on Behavior of Corroded RC Columns Subjected to Lateral Loads ... 37

2.4.2  Studies on Lateral Behavior of RC Corroded Elements .......................... 42

2.5  Finite Element Modeling of RC Columns .................................................. 45

2.6  Analytical Models to Estimate Lateral Capacity of Corroded RC Members .... 49

2.7  Summary and Conclusion ........................................................................ 52

3  FINITE ELEMENT MODELING .................................................................... 54

3.1  Introduction ................................................................................................ 54

3.2  Element Types and Material Properties .................................................. 54

3.2.1  Concrete ................................................................................................ 55

3.2.2  Reinforcing Steel .................................................................................... 59

3.2.3  Rigid Plate ............................................................................................. 63

3.2.4  Contact Element .................................................................................... 63

3.3  Loading ...................................................................................................... 63
3.4 Nonlinear Static Analysis ................................................................. 64
3.5 Validation of Experimental Data ...................................................... 64
   3.5.1 Modeling of Corroded Elements .............................................. 65
      3.5.1.1 Maaddawy et al. (2005) ................................................. 65
      3.5.1.2 Gong (2009) ................................................................. 70
      3.5.1.3 Ou et al. (2012) ............................................................. 77
      3.5.1.4 Meda et al. (2014) ......................................................... 82
3.6 Summary and Conclusion .............................................................. 88

4 LATERAL STRENGTH EVALUATION OF CORRODED COLUMNS .............. 89
   4.1 Introduction ................................................................................. 89
   4.2 Objectives and Scopes ................................................................. 89
   4.3 Finite Element Analysis of Corroded Columns .............................. 93
      4.3.1 Compression-side Corroded Columns .................................. 93
      4.3.2 Tension-side Corroded Columns ......................................... 99
      4.3.3 All-sides Corroded Columns .............................................. 106
   4.4 Summary and Conclusion ........................................................... 113

5 EFFECT OF DIFFERENT PARAMETERS ON LATERAL CAPACITY OF
SEVERELY CORRODED RC COLUMNS ...................................................... 116
   5.1 Introduction ................................................................................. 116
   5.2 Compression-side Corroded Columns .......................................... 116
      5.2.1 Effect of Corrosion Level .................................................... 116
5.2.2 Effect of Length of Corroded Zone ........................................ 118
5.2.3 Effect of Compressive Strength of Concrete .......................... 120
5.2.4 Effect of Steel Reinforcing Ratio ($\rho$) ................................. 121
5.2.5 Effect of Axial Load .......................................................... 123
5.2.6 Effect of Shear Span to Depth Ratio (L/d) ......................... 124

5.3 Tension-side Corroded Columns .......................................... 124
5.3.1 Effect of Corrosion Level .................................................. 125
5.3.2 Effect of Length of Corroded Zone .................................... 126
5.3.3 Effect of Compressive Strength of Concrete ...................... 128
5.3.4 Effect of Steel Reinforcing Ratio ($\rho$) ................................. 129
5.3.5 Effect of Axial Load .......................................................... 129
5.3.6 Effect of Shear Span to Depth Ratio (L/d) ......................... 131

5.4 All-sides Corroded Columns ............................................. 131
5.4.1 Effect of Corrosion Level .................................................. 132
5.4.2 Effect of Length of Corroded Zone .................................... 133
5.4.3 Effect of Compressive Strength of Concrete ...................... 135
5.4.4 Effect of Steel Reinforcing Ratio ($\rho$) ................................. 136
5.4.5 Effect of Axial Load .......................................................... 138
5.4.6 Effect of Shear Span to Depth Ratio (L/d) ......................... 139

5.5 Summary and Conclusion ................................................ 140
6 PRACTICAL MODEL TO ESTIMATE THE LATERAL CAPACITY OF SEVERELY CORRODED RC COLUMNS ................................................................................................................................. 142

6.1 Introduction .................................................................................................................. 142

6.2 Methodology .................................................................................................................. 142

6.2.1 Lateral Capacity of Fully-bonded Corroded Columns ............................................. 143

6.2.2 Lateral Capacity Reduction Factor $a_{corr}$ ............................................................... 145

6.3 Practical Model for Compression-side Corroded Columns ....................................... 145

6.4 Practical Model for Tension-side Corroded Columns ................................................... 148

6.5 Practical Model for All-sides Corroded Columns ......................................................... 149

6.6 Discussion on Corroded Columns with Flexural-shear or Shear Failure ............... 151

6.7 Summary and Conclusion .......................................................................................... 153

7 SUMMARY AND CONCLUSIONS ..................................................................................... 155

7.1 General Conclusions .................................................................................................... 156

7.2 Conclusions for Compression-side Corroded Columns ............................................. 157

7.3 Conclusions for Tension-side Corroded Columns ......................................................... 158

7.4 Conclusions for All-sides Corroded Columns ............................................................... 158

7.5 Limitations and Recommendations for Future Study ............................................... 159

APENDIX 1: LATERAL LOAD- DISPLACEMENT CURVES OF COMPRESSION-SIDE CORRODED COLUMNS ......................................................................................................................... 161

APENDIX 2: LATERAL LOAD- DISPLACEMENT CURVES OF TENSION-SIDE CORRODED COLUMNS ................................................................................................................................. 210
APENDIX 3: LATERAL LOAD- DISPLACEMENT CURVES OF ALL-SIDES

CORRODED COLUMNS ........................................................................................................... 261

REFERENCES......................................................................................................................... 310

VITEA ..................................................................................................................................... 319
LIST OF FIGURES

Figure 2-1 Effect of steel corrosion on concrete structures (CONTECVET, 2001)................................. 6
Figure 2-2 carbonation-induced and chloride-induced corrosions (CONTECVET, 2001) ................... 10
Figure 2-3 residual section of corroded bar (Gonzalez, 1994) ................................................................. 13
Figure 2-4 decrease of bar sectional area for \[ l_{corr} = 1 \mu A/cm^2 \] (CONTECVET, 2001) ............... 13
Figure 2-5 stress-strain relationship of sound and corroded steel bars (Lee and Cho, 2009) ........ 16
Figure 2-6 Constitutive law for cracked concrete in compression .......................................................... 18
Figure 2-7 crack width due to corrosion at: (a) longitudinal bars, (b) transverse bars ..................... 19
Figure 2-8 Stress-strain diagram for cracked concrete cover (CEB-FIP, 1990) ................................. 20
Figure 2-9 Concrete cover due to corrosion cracking process (Barghava et al., 2006) ................. 21
Figure 2-10 Bond-slip relationship (CEB-FIP90) ................................................................................. 24
Figure 2-11 Gan's model for bond-slip relationship (Gan, 2000) ....................................................... 26
Figure 2-12 Cyclic bond-slip curve for Pullout test (Eligehausen et al., 1983) ................................. 27
Figure 2-13 Effect of number of cycles on bond-slip curve (Eligehausen et al., 1983) ................. 27
Figure 2-14 Deterioration of concrete bridge pier columns due to corrosion of bars (Aboutaha, 2004) ................................................................................................................. 31
Figure 2-15 Close-up of corroded longitudinal and transverse bars of a column (Aboutaha, 2004) 31
Figure 2-16: Effect of corrosion of reinforcing steel bars on the surrounding concrete (Aboutaha, 2004) .................................................................................................................................. 32
Figure 2-17: Corrosion damaged rectangular concrete columns (Aboutaha, 2004) ......................... 33
Figure 2-18 Corrosion damaged circular concrete columns. Notice that the main cracks are parallel to the columns’ main reinforcing steel bars (Aboutaha, 2004) ......................................................... 34
Figure 2-19 Corroded longitudinal and transverse bars for circular columns (Aboutaha, 2004) ....... 35
Figure 2-20 Deteriorated concrete bridge pier due to corrosion of bars (Aboutaha, 2004) ............ 35
Figure 2-21 Corrosion damage of concrete bridge pier columns (Aboutaha, 2004) .......................... 36

Figure 2-22 Load-deformation curves (influence of rebar corrosion) (RC-COR-1: 1st corrosion level, RC-COR-2: 2nd corrosion level, RC-COR-3: 3rd corrosion level) (Lee at al., 2003) ....................... 38

Figure 2-23 Lateral load-displacement curve (Ma et al., 2012) .............................................................. 40

Figure 2-24 Load-deformation curves of un-corroded vs. corroded column (Meda et al., 2014) .......... 41

Figure 2-25 Crack pattern, (a) to (e): increase in corrosion rate from, (f) close view of fracture of transverse reinforcement for (e) (Ou et al., 2012) ............................................................... 45

Figure 2-26 Plan view of concrete cracking in beam web due to corrosion for three different stirrup spacing (Higgins et al., 2003) ...................................................................................... 51

Figure 3-1 Continuum (solid) element geometry (ABAQUS 6.14) .......................................................... 55

Figure 3-2 Concrete model in compression for $f' = 4000$ psi ................................................................. 57

Figure 3-3 Concrete model in tension for $f' = 4000$ psi ........................................................................ 58

Figure 3-4 Beam-type element geometry (ABAQUS 4.13) ................................................................. 59

Figure 3-5 Tensile stress-strain diagram of un-corroded and corroded steel (Grade 60) .......... 61

Figure 3-6 Buckling stress diagram of corroded steel (Grade 60) ....................................................... 62

Figure 3-7 Large displacement load-deflection behavior of a pinned-ended column (Chen & Lui, 1987) .......................................................................................................................... 62

Figure 3-8 Boundary conditions (a), axial force loading (b), and lateral displacement loading (c) of modeled column ............................................................................................................. 64

Figure 3-9 Geometry of Maaddawy's test beam .................................................................................. 65

Figure 3-10 Lateral Load-displacement plot and Crack pattern of Maaddawy's test beam (Virgin) . 67

Figure 3-11 Lateral force-lateral displacement curves of Maaddawy's test sound (virgin); experimental vs. FEM model ........................................................................................................... 68

Figure 3-12 Lateral Load-displacement plot and Crack pattern of Maaddawy's test beam CN-310 69
Figure 3-13 Lateral force-lateral displacement curves of Maaddawy’s test beam CN-310; experimental vs. FEM model ...........................................................................................................................69

Figure 3-14 Geometry of Gong’s test column .......................................................................................................................... 71

Figure 3-15 Lateral Load-displacement plot and Crack pattern of Gong’s test column A0 ..........................72

Figure 3-16 Lateral force-lateral displacement curves of Gong’s test columns A0; experimental vs. FEM model .................................................................................................................................... 73

Figure 3-17 Lateral Load-displacement plot of Gong’s test column B3 ......................................................... 73

Figure 3-18 Lateral force-lateral displacement curves of Gong’s test columns B3; experimental vs. FEM model .................................................................................................................................... 74

Figure 3-19 Lateral Load-displacement plot of Gong’s test column C2 ......................................................... 75

Figure 3-20 Lateral force-lateral displacement curves of Gong’s test columns C2; experimental vs. FEM model .................................................................................................................................... 76

Figure 3-21 Condition of Gong’s test column after corrosion.............................................................................. 76

Figure 3-22 Geometry of Ou’s test beam .......................................................................................................................... 78

Figure 3-23 Lateral Load-displacement plot and Crack pattern of Ou’s test beam B-0 .........................79

Figure 3-24 Lateral force-lateral displacement curves of Ou’s test beam B-0; experimental vs. FEM model .................................................................................................................................... 79

Figure 3-25 Lateral Load-displacement plot of Ou’s test beam B-150 ......................................................... 80

Figure 3-26 Lateral force-lateral displacement curves of Ou’s test beam B-150; experimental vs. FEM model .................................................................................................................................... 81

Figure 3-27 Condition of Ou’s test beam after corrosion and loading ................................................................. 81

Figure 3-28 Geometry of Meda’s test column .................................................................................................................. 83

Figure 3-29 Lateral Load-displacement plot and Crack pattern of Meda’s test column UC .................. 84

Figure 3-30 Lateral force-lateral displacement curves of Meda’s test columns UC; experimental vs. FEM model .................................................................................................................................... 85
Figure 3-31 Lateral Load-displacement plot of Meda's test column CC ........................................... 85
Figure 3-32 Lateral force-lateral displacement curves of Meda's test column CC; experimental vs.
FEM model ................................................................................................................................. 86
Figure 3-33 Condition of Meda's test column after corrosion .......................................................... 87
Figure 3-34 Lateral force-lateral displacement curves of Meda's test column CC; experimental vs.
FE models of Vu et al. and current study ....................................................................................... 87
Figure 3-35 Comparison of lateral capacity of corroded elements, Experimental vs. FEA results .... 88
Figure 4-1 (a) Side View of the Column; (b) Cross-section of the Column ......................................... 90
Figure 4-2 Cases Studied in Finite Element Analysis ......................................................................... 92
Figure 4-3 Lateral capacity- displacement diagram of compression-side corroded column compared
to un-corroded column (L/d=5) .................................................................................................... 95
Figure 4-4 Crack pattern of un-corroded and compression-side corroded columns (L/d=5); (a to c)
respectively axial load ratios of 5%, 15% and 25%. ........................................................................ 96
Figure 4-5 Lateral capacity- displacement diagram of compression-side corroded column compared
to un-corroded column (L/d=2.5) ............................................................................................... 97
Figure 4-6 Crack pattern of un-corroded and compression-side corroded columns (L/d=2.5); (a to c)
respectively axial load ratios of 5%, 15% and 25%. ........................................................................ 98
Figure 4-7 Lateral capacity- displacement diagram of tension-side corroded column compared to
un-corroded column (L/d=5) ......................................................................................................... 100
Figure 4-8 Crack pattern of un-corroded and tension-side corroded columns (L/d=5); (a to c)
respectively axial load ratios of 5%, 15% and 25%. ....................................................................... 102
Figure 4-9 Lateral capacity- displacement diagram of tension-side corroded column compared to
un-corroded column (L/d=2.5) ..................................................................................................... 104
Figure 4-10 Crack pattern of un-corroded and tension-side corroded columns (L/d=2.5); (a to c)
respectively axial load ratios of 5%, 15% and 25%. ....................................................................... 105
Figure 4-11 Lateral capacity-displacement diagram of all-sides corroded column compared to un-corroded column (L/d=5) .......................................................... 107

Figure 4-12 Crack pattern of un-corroded and all-sides corroded columns (L/d=5); (a to c) respectively axial load ratios of 5%, 15% and 25%. ......................................................... 109

Figure 4-13 Lateral capacity-displacement diagram of all-sides corroded column compared to un-corroded column (L/d=2.5) .......................................................... 111

Figure 4-14 Crack pattern of un-corroded and all-sides corroded columns (L/d=2.5); (a to c) respectively axial load ratios of 5%, 15% and 25%. ......................................................... 113

Figure 5-1 Effect of corrosion level and length of corroded zone on lateral capacity of compression-side corroded columns (L/d=5) .......................................................... 117

Figure 5-2 Effect of corrosion level and length of corroded zone on lateral capacity of compression-side corroded columns (L/d=2.5) .......................................................... 118

Figure 5-3 Effect of length of corroded zone on lateral capacity of compression-side corroded columns (L/d=5) .......................................................... 119

Figure 5-4 Effect of length of corroded zone on lateral capacity of compression-side corroded columns (L/d=2.5) .......................................................... 120

Figure 5-5 Effect of $f'_c$ on lateral capacity reduction of compression-side corroded columns (L/d=5) .......................................................... 121

Figure 5-6 Effect of $\rho$ on lateral capacity reduction of compression-side corroded columns (L/d=5) .......................................................... 122

Figure 5-7 Effect of $\rho$ on lateral capacity reduction of compression-side corroded columns (L/d=2.5) .......................................................... 122

Figure 5-8 Effect of axial load on lateral capacity reduction of compression-side corroded columns .......................................................... 123

Figure 5-9 Effect of L/d on lateral capacity reduction of compression-side corroded columns ...... 124
Figure 5-10 Effect of corrosion level and length of corroded zone on lateral capacity of tension-side corroded columns (L/d=5) .......................................................... 125

Figure 5-11 Effect of corrosion level and length of corroded zone on lateral capacity of tension-side corroded columns (L/d=2.5) .......................................................... 126

Figure 5-12 Effect of corrosion level on lateral capacity of tension-side corroded columns (L/d=5) .......................................................................................................................... 126

Figure 5-13 Effect of length of corroded zone on lateral capacity of tension-side corroded columns (L/d=5) .......................................................................................................................... 127

Figure 5-14 Effect of length of corroded zone on lateral capacity of tension-side corroded columns (L/d=2.5) .......................................................................................................................... 128

Figure 5-15 Effect of $f'c$ on lateral capacity reduction of tension-side corroded columns (L/d=5) 129

Figure 5-16 Effect of axial load on lateral capacity reduction of tension-side corroded columns ... 130

Figure 5-17 Effect of L/d on lateral capacity reduction of tension-side corroded column ............ 131

Figure 5-18 Effect of corrosion level and length of corroded zone on lateral capacity of all-sides corroded columns (L/d=5) .......................................................................................................................... 132

Figure 5-19 Effect of corrosion level and length of corroded zone on lateral capacity of all-sides corroded columns (L/d=2.5) .......................................................................................................................... 133

Figure 5-20 Effect of length of corroded zone on lateral capacity of all-sides corroded columns (L/d=5) .......................................................................................................................... 134

Figure 5-21 Effect of length of corroded zone on lateral capacity of all-sides corroded columns (L/d=2.5) .......................................................................................................................... 135

Figure 5-22 Effect of $f'c$ on lateral capacity reduction of all-sides corroded columns (L/d=5) .... 136

Figure 5-23 Effect of $\rho$ on lateral capacity reduction of all-sides corroded columns (L/d=5) .... 137

Figure 5-24 Effect of $\rho$ on lateral capacity reduction of all-sides corroded columns (L/d=2.5) .... 137

Figure 5-25 Effect of axial load on lateral capacity reduction of all-sides corroded columns ....... 138
Figure 5-26 Effect of L/d on capacity reduction of compression-side corroded columns ............. 139

Figure 6-1 Proposed $acorr$ reduction factor for compression-side corroded columns with L/d=5

\[ f'c = 4 \text{ ksi} \] ............................................................................................................................. 146

Figure 6-2 Proposed $acorr$ reduction factor for compression-side corroded columns with L/d=5

\[ f'c = 7 \text{ ksi} \] ............................................................................................................................. 147

Figure 6-3 Comparison of lateral capacity of compression-side corroded columns based on

proposed practical model with FEA results ......................................................................................... 147

Figure 6-4 Proposed $acorr$ reduction factor for tension-side corroded columns with L/d=5........ 148

Figure 6-5 Proposed $acorr$ reduction factor for all-sides corroded columns with L/d=5 and $f'c = 4 \text{ ksi}$ ................................................................................................................................. 150

Figure 6-6 Proposed $acorr$ reduction factor for all-sides corroded columns with L/d=5 and $f'c = 7 \text{ ksi}$ ................................................................................................................................. 150

Figure 6-7 Comparison of lateral capacity of all-sides corroded columns based on proposed

practical model with FEA results ........................................................................................................ 151

Figure 6-8 Comparison of shear capacity of experimental tests and proposed ACI equations (ACI-

ASCE Committee 426, 1978) ............................................................................................................. 152

Figure 6-9 Comparison of lateral capacity of corroded columns failed in shear mode based on

proposed practical model with FEA results; (a) tension-side corroded columns, (b) compression-side corroded columns, (c) all-sides corroded columns .................................................... 153
LIST OF TABLES

Table 2-1 Relationship between corrosion current and corrosion level........................................... 11
Table 2-2 Parameters to define mean bond-slip equation................................................................. 24
Table 2-3 Relation between corrosion rate of longitudinal and transverse bars ......................... 53
Table 3-1 Concrete models in compression .................................................................................. 56
Table 3-2 Concrete models in tension .......................................................................................... 58
Table 3-3 Reinforcing Steel Model for Un-corroded bars ......................................................... 60
Table 3-4 Compressive Reinforcing Steel Model for Corroded Bars ........................................ 61
Table 3-5 Geometry and material properties of Maaddawy’s test beams .................................. 66
Table 3-6 Comparison between experimental and FEA results; Maaddawy’s test beam (Virgin) .... 68
Table 3-7 Comparison between experimental and FEA results; Maaddawy’s test beam CN-310 ..... 70
Table 3-8 Geometry and material properties of Gong’s test columns ......................................... 71
Table 3-9 Comparison between experimental and FEA results; Gong’s test column A0 ............. 72
Table 3-10 Comparison between experimental and FEA results; Gong’s test column B3 .......... 74
Table 3-11 Comparison between experimental and FEA results; Gong’s test column C2 .......... 75
Table 3-12 Geometry and material properties of Ou’s test beams ............................................. 78
Table 3-13 Comparison between experimental and FEA results; Ou’s test beam B-0 ................. 79
Table 3-14 Comparison between experimental and FEA results; Ou’s test beam B-150 .......... 80
Table 3-15 Geometry and material properties of Meda’s test columns ....................................... 83
Table 3-16 Comparison between experimental and FEA results; Meda’s test column UC .......... 84
Table 3-17 Comparison between experimental and FEA results; Meda’s test column CC .......... 86
Table 5-1 Lateral capacity ratio of corroded columns to un-corroded columns ($V_{corr}/V_{UC}$) .... 141
1 INTRODUCTION

1.1 Background

Corrosion of reinforcing steel bars is the primary durability problem that causes degradation of reinforced concrete structures located in aggressive environments. Severe corrosion of steel bars decreases the lateral load-carrying capacity of reinforced concrete members, causes loss in the mechanical properties of reinforcement and cross-sectional area of steel bars and concrete cover, bond deterioration, reduces anchorage of steel bars, and decreases the confinement by transverse reinforcement. Consequently, corrosion results in drop in the lateral strength of columns. Therefore, studying response of corroded reinforced concrete columns subjected to lateral loads is necessary.

According to the Federal Highway Administration (FHWA) report in 2013, 25.9 percent of the total inventory of highway bridges are deficient or functionally obsolete. Corrosion damage caused by deicing salts is considered one of the main problems that cause a bridge structure to be structurally deficient (FHWA, 2004). However, absence of a practical model for assessment of the residual lateral strength of corroded RC columns as well as the lack of research on ultimate lateral capacity of deteriorated concrete structures shows the need to develop a practical method to calculate the current lateral capacity of corroded reinforced concrete bridge columns. This study investigates response of severely corroded steel reinforced concrete columns subjected to lateral loading and axial compressive load by 1) finite element analyzing of about 310 specimens, 2) studying effect of different parameters on them and 3) proposing a practical model to estimate their lateral capacity.
1.2 Objectives and Scopes

The major type of RC bridge deterioration is corrosion of reinforcement. Severe corrosion of steel bars decreases the lateral load-carrying capacity of reinforced concrete members, causes loss in the mechanical properties of reinforcement and cross-sectional area of steel bars and concrete cover, bond deterioration, reduces anchorage of steel bars, and decreases the confinement by transverse reinforcement. Due to lack of studies on residual lateral capacity of columns after severe corrosion, it is necessary to study the behavior of corroded columns, especially columns subjected to combined axial load, bending moment and shear force. Better understanding of the structural performance of corroded columns can lead to enhanced inspection procedures and development of cost-effective and strategic rehabilitation methods.

The objective of this research is to develop a practical method for evaluating the existing lateral strength of corroded RC columns, which can be adapted to existing bridge condition evaluation methods. The proposed method considers all the changes in material and geometry properties including area loss of corroded steel bars and concrete cover, bond deterioration and its consequences of corroded bars’ buckling, corrosion location in the column, corrosion length along the column, compressive strength of concrete, reinforcing ratio of RC column section, axial load ratio, and shear span to depth ratio. This study also provides engineers a better understanding of lateral response of severely corroded RC bridge columns with detailed force-displacement diagrams based on finite element analysis.
1.3 Research Approach

The general plan for this project is as follows:

- Review of literature on effect of the various variables on the lateral strength of corroded steel reinforced concrete columns;
- Compilation of experimental research data of un-corroded columns and corroded beams and columns;
- Development of a Finite Element Model for corroded concrete columns with ABAQUS;
- Verification of the FE model using the limited compiled experimental research data;
- Studying the effect of different variables on lateral capacity of corroded columns;
- Development of a practical analysis model for estimating the lateral strength of corroded reinforced concrete column.

1.4 Outline

Chapter 1 introduces the background of the problem, objectives and scope of the dissertation and the research approach.

Chapter 2 reviews the literature, including the general effects of corrosion on corroded RC members, structural behavior of corroded reinforced concrete elements, experimental studies and finite element modeling of corroded RC members and analytical models to estimate the lateral capacity of RC elements.

Chapter 3 shows finite element modeling of the corroded columns in ABAQUS, from
introducing the element types and material properties to loading and analysis. Finally, the experimental data is validated based on finite element modeling of experimental specimens.

Chapter 4 presents the results of finite element analysis for 308 specimens and detailed study of lateral behavior of corroded RC columns when the location of corrosion is at tension-side, compression side and all-sides of the column section.

Chapter 5 studies the effect of different parameters on lateral response of severely corroded RC column. These parameters include the location of corrosion on cross section of column, shear span to depth ratio, corrosion level, corrosion height, compressive strength of the concrete, reinforcing ratio and axial load ratio.

Chapter 6 proposes a practical model to estimate the residual lateral capacity of severely corroded RC columns for each of the three cases (i.e. compression-side, tension-side and all-sides corroded column,) including all variables.

Chapter 7 presents the summary and conclusions drawn from this study. Limitations and recommendations for future studies are also included in this chapter.
2 LITERATURE REVIEW

2.1 Introduction

According to the Federal Highway Administration (FHWA) report "Status of the Nation's Highways, Bridges, and Transit: 2013 Conditions and Performance", overall, approximately 11.7 percent of bridges were classified as structurally deficient in 2010, and 14.2 percent were classified as functionally obsolete. This represents 25.9 percent of the total inventory of highway bridges when bridges are weighted equally. According to the FHWA report, corrosion damage caused by deicing salts is considered one of the main problems that cause a bridge structure to be structurally deficient (FHWA, 2004).

Corrosion of reinforcing steel bars is the primary durability problem that causes degradation of reinforced concrete structures located in aggressive environments (Aquino and Hawkins, 2007). If the rate of corrosion is high, it may reduce lateral load-carrying capacity of reinforced concrete members, cause bond deterioration, reduce anchorage of steel bars, and decrease the confinement by transverse reinforcement (Ma et al, 2012).

The primary cause of collapse in many existing older structures is column failure. Therefore, studying response of corroded reinforced concrete columns subjected to lateral loads is necessary.

This chapter presents background and literature review of reinforced concrete elements, especially columns, subjected to corrosion. Firstly, corrosion issues such as general effects of corrosion on RC concrete structures and structural behavior of corroded reinforced concrete elements are investigated. Secondly, experimental studies on un-corroded columns and corroded
beams and columns are explored. Finally, modeling of RC columns including finite element modeling of corroded RC members and analytical modeling of RC elements are presented.

2.2 General Effects of Corrosion on RC Concrete Structures

Corrosion of reinforcing steel bars is the primary durability problem that causes degradation of reinforced concrete structures located in aggressive environments (Aquino and Hawkins, 2007). Basically three factors affect structural performance of reinforced concrete members with corroded reinforcements (Morinaga, 1996) & (Nakayama et al., 1995):

- Losses in the mechanical performance of reinforcing bars due to the losses in their cross-sectional area and ductility,
- Losses in the effective cross-sectional area of concrete due to cracking in the cover concrete,
- Losses in the bond performance of concrete with reinforcements.

However, reduction in confinement by rebar, exposed length of corroded steel bars, loss of symmetry in steel bars and cross-sectional asymmetry are some other factors that influence

![Figure 2-1 Effect of steel corrosion on concrete structures (CONTECVET, 2001)](image-url)
load carrying capacity of the deteriorated reinforced concrete members (Tapan, 2008).

In order to investigate the structural performance of corroded columns, it is necessary to review the mechanism of corrosion first to understand how corrosion affects function of reinforced concrete elements in detail.

### 2.2.1 Mechanism of Corrosion

Steel reacts with oxygen or water in the atmosphere to produce lower natural status oxide or hydroxide (Hansson et al., 2007).

\[
Fe \rightarrow Fe^{++} + 2e^- \quad \text{(Anodic Reaction)} \quad (1)
\]

\[
\frac{1}{2}O_2 + H_2O + 2e^- \rightarrow 2OH^- \quad \text{(Cathodic Reaction)} \quad (2)
\]

The embedded steel in concrete is protected by concrete itself. As the cement hydrates in fresh concrete, the pH of the mixing water increases.

\[
\begin{align*}
\{2(3CaO, SiO_2) + 6H_2O & \rightarrow 3CaO, 2SiO_2, 3H_2O + 3Ca(OH)_2 \\
2(CaO) + 3H_2O & \rightarrow 2Ca(OH)_2
\}
\end{align*}
\]

Well hydrated cement may contain 15% to 30% calcium hydroxide, \( Ca(OH)_2 \), by weight of original cement; which provides alkality of 12 to 13 pH (Aboutaha, 2004).

The corrosion products in high pH levels produce a thin protective passive film \( Fe_2O_3 \) on the surface of the steel, which prevents iron ions \( Fe^{++} \) from entering the concrete and does not allow oxygen anions \( O^- \) within the concrete to contact the steel surface (Aboutaha, 2004). Therefore, the thin film limits steel degradation due to corrosion to about 0.1-1.0 μm/year. The
passive corrosion does not cause significant degradation of steel within a 75 year lifetime (Hansson et al. 2007).

The passive film breaks down in the presence of chloride ions or when the pH level of concrete drops below 9. Chloride ions from de-icing salts or at chloridic environments, can penetrate the permeable concrete easily. The concrete also tends to react with acidic gasses, especially $CO_2$ to neutralize the alkali pore solution to reduce the pH level. Active corrosion, either due to chloride ions or carbonation, starts to take place as soon as the passive film is destroyed. Steel degradation due to active corrosion can increase damages of reinforced concrete elements to high levels of several mm/year (Hansson et al., 2007).

2.2.1.1 Chloride-induced Corrosion

Chloride ions exist in concrete, either by initial destructive components of concrete mixture or due to external environmental ionic attack. Initial concrete mixture may have chlorides in aggregates, water or admixtures. Chloride ions can enter the concrete electrolyte in coastal regions when exposed to sea spray. Use of de-icing salts on roads and bridges also causes chloride ions to penetrate concrete (Aboutaha, 2004).

The amount of chloride ions in concrete has a great effect on the passive film of the steel, regardless of the level of alkalinity of the concrete.

Forming of passive film over steel reinforcement in alkali concrete makes the steel bars be in passive zone. Chloride ions in concrete tend to concentrate on a part of steel to activate chemical reactions, while unexposed parts are still passive. Non-uniform distribution of chloride ions causes different potentials that initiate flow of electrons between them, which can be along the steel bar or within the concrete thickness.
Concentration of chloride ions at the anode results in production of more electrons for reactions at the cathode and destruction of the passive region.

\[ Fe + 2Cl^- \rightarrow FeCl_2 + 2e^- \]  \hspace{1cm} (4)

\[ FeCl_2 + 2H_2O \rightarrow Fe(OH)_2 + 2H^+ + 2Cl^- \]  \hspace{1cm} (5)

“These reactions clearly illustrate the continuous role chloride ions have in the corrosion process as the chloride ion is released at the cathode and used again in the anodic reaction” (Aboutaha, 2004).

### 2.2.1.2 Carbonation-induced Corrosion

Carbonation is the reaction of carbon dioxide \((CO_2)\) from the atmosphere with the calcium hydroxide \((Ca(OH)_2)\) existing in the cement of the concrete, which results in the reduction of alkalinity in concrete. Carbonation appears on the surface of concrete.

\[ Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \]  \hspace{1cm} (6)

In order for the above reaction to take place, the hydroxyl ions are removed from pore water solution in the concrete, therefore pH reduces from initial value of 12 to 8 (Tapan, 2008).

Carbonation initiates at the surface of concrete. As the depth of carbonation reaches the reinforcing bars, because of low alkalinity, the passive film becomes unstable and active corrosion starts. Corrosion due to carbonation is usually general and homogeneous. The corrosion products of carbonation in the early stages, usually appears as rust stains on the surface of concrete without causing cracks on concrete cover (Hansson et al. 2007).
The rate of carbonation is very low in normal circumstances; however, under a specific range of relative humidity, a large amount of carbon dioxide can penetrate into the concrete pores and react with calcium hydroxide. The highest speed of carbonation happens at relative humidity of 50-70%. However, the most destructive environment for active corrosion is alternate semi-dry and wet cycles (Tuutti, 1980).

In general, carbonation-induced corrosion has a descending progress. Carbon dioxide needs to penetrate into concrete further, while concrete becomes less permeable. As the concrete ages, it hydrates more. Also more existing water, produced during carbonation accelerates hydration of concrete. On the other hand, carbonates settle in concrete pores. All these facts make the concrete impermeable (Nielsen, 1985).

As shown in Figure 2-2, the main development of corrosion is done by micro-cell actions between short distance of anode and cathode. Because of the relatively far distances in localized corrosion, macro-cell corrosion takes place (CONTECVET, 2001).

![Figure 2-2 carbonation-induced and chloride-induced corrosions (CONTECVET, 2001)](image-url)
2.2.2 Corrosion Rate

The quantity of oxidized steel can be measured by corrosion current, $I_{corr}$. Using a reference electrode as the electrical potential and a secondary electrode as a current producer, corrosion current is measured. Polarization resistance technique, $R_p$, is the most common method to determine corrosion current. $R_p$ is based in very small polarization around the corrosion potential (CONTECVET, 2001):

$$R_p = \frac{\Delta E}{\Delta I}, \Delta E < 20 \text{ mV}$$  \hspace{1cm} (7)

$$I_{corr} = \frac{B}{R_p}$$  \hspace{1cm} (8)

The value of 26 mV is usually taken for constant B. Table 2-1 presents the relationship between corrosion current and level of corrosion according to manual of (CONTECVET, 2001).

<table>
<thead>
<tr>
<th>$I_{corr}$ ((\mu\text{m/year or } \frac{\mu\text{A}}{\text{cm}^2}/11.6))</th>
<th>Level of Corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1</td>
<td>Negligible</td>
</tr>
<tr>
<td>1-5</td>
<td>Low</td>
</tr>
<tr>
<td>5-10</td>
<td>Moderate</td>
</tr>
<tr>
<td>&gt;10</td>
<td>High</td>
</tr>
</tbody>
</table>

Corrosion rate, CR, is defined as the average mass loss of the reinforcing bars:

$$CR = \frac{\Delta w}{w} \times 100$$  \hspace{1cm} (9)

Where; $\Delta w$ is the average mass loss of corroded bars and $w$ is the mass of original bars.

The above equation can be presented in terms of the average cross-sectional area, if corrosion is uniform.

$$CR = \frac{\Delta A_{sec}}{A_{sec}} \times 100 = \left(\frac{\Delta d}{d_i}\right)^2 \times 100$$  \hspace{1cm} (10)
Where; \( d_i \) is the diameter of un-corroded reinforcing bar. The reduced diameter of corroded bar can be defined according to section.

Corrosion rate has been also proposed in terms of corrosion current and nominal diameter of reinforcing bar as following (Du et al., 2005):

\[
CR = 0.046 \frac{l_{corr}}{d_i} t \times 100
\]

(11)

Where; \( d_i \) is the nominal diameter of un-corroded reinforcing bar in mm, \( l_{corr} \) is the corrosion current in \( \mu A/cm^2 \) and \( t \) is the time in years. Further explanations on this equation are presented in section 2.2.3.1.

2.2.3 Corrosion Effect on Mechanical Properties of Steel Bars

Mechanical properties of steel reinforcing bars including bar cross-sectional area and strength and ductility of steel reduce due to corrosion.

2.2.3.1 Bar Cross Sectional Area Loss due to Corrosion

Reduction in cross-sectional area of steel reinforcing bars due to corrosion depends on corrosion type. Carbonation and chloride ions have different effects on bars, causing homogeneous (uniform) and localized (pitting) corrosion, respectively. Figure 2-3 shows residual section of corroded bar in two different aggressive agents.

The reduced diameter of corroded steel bar is a function of nominal diameter and the attack penetration factor (Gonzalez, 1994).

\[
d_t = d_i - \alpha P_x
\]

(12)

Where; \( d_t \) is diameter of corroded bar in mm and \( d_i \) is the nominal diameter of un-
corroded bar in mm. \( \alpha \) is equal to 2 for carbonation-induced corrosion and up to 10 for chloride-induced (localized) corrosion. \( P_x \), the attack penetration factor, is the average value of corrosion penetration (bar radius reduction) in mm.

\[
P_x = 0.0115 l_{corr} t
\]  

(13)

Which depends on time \( t \) (years) and \( I_{corr} \), corrosion current (\( \mu A/cm^2 \)).

As the bar diameter increases, the homogeneous corrosion has less effect on diameter reduction of corroded steel bar than pitting corrosion.

![Figure 2-3 residual section of corroded bar (Gonzalez, 1994)](image)

![Figure 2-4 decrease of bar sectional area for \( I_{corr} = 1 \mu A/cm^2 \) (CONTECVET, 2001)](image)
The experimental tests conducted by Du et al. (2005) shows that the residual cross-section of a corroded bar is no longer round and varies considerably along its length and its circumference because of local attack penetration. Practically, they considered $\alpha$, at the above equation, equal to 4; then the residual diameter of the corroded reinforcing bar can be calculated knowing the corrosion rate.

$$CR = \frac{\alpha P_x}{d_i} \times 100 = 0.046 \frac{I_{corr}}{d_i} t \times 100$$  \hspace{1cm} (14)$$

$$CR = [1 - \left(\frac{d_i}{d_i}\right)^2] \times 100$$  \hspace{1cm} (15)$$

Then average cross sectional area of corroded bar ($A_{corr}$) is defined knowing the initial cross sectional area of un-corroded bar ($A_0$) and the corrosion rate (Du et al., 2005):

$$A_{corr} = A_0 (1 - 0.01 CR)$$  \hspace{1cm} (16)$$

### 2.2.3.2 Strength and Ductility Reduction due to Corrosion

A few studies were carried out to investigate the stress-strain relationship of corroded steel reinforcing bars. Du et al. (2005) conducted accelerated and simulated corrosion tests on both bare bars and embedded bars in concrete to determine how much corrosion can reduce strength and ductility of steel bars. Corrosion does not affect modulus of elasticity, hardening strain and strength ratio ($f_u/f_y$), substantially. However, there is a considerable drop in yielding strength and ultimate strain of corroded steel bars.

$$f_{y,corr} = f_{y0} (1 - 0.005 CR)$$  \hspace{1cm} (17)$$

$$\varepsilon_{u,corr} = \varepsilon_{u0} (1 - 0.05 CR)$$  \hspace{1cm} (18)$$
Lee and Cho (2009) conducted experiments on steel reinforcements using chloride-induced corrosion and electrical current to study the effect of pitting and uniform corrosion on mechanical properties of steel bars. Assuming the cross-sectional area of corroded bar is the same as cross-sectional area of sound bar, they proposed the following equations for yielding strain ($\varepsilon_{y_{corr}}$), ultimate stress ($f_{u_{corr}}$) and elastic modulus ($E_{corr}$). Obviously, for the same rate of corrosion, pitting corrosion affects the mechanical properties of steel more than uniform corrosion.

For uniform corrosion:

\begin{align*}
\varepsilon_{y_{corr}} &= \varepsilon_{y_0}(1 - 0.0124CR) \\
nf_{u_{corr}} &= nf_{u_0}(1 - 0.0107CR) \\
E_{corr} &= E_0(1 - 0.0075CR)
\end{align*}

For pitting corrosion:

\begin{align*}
\varepsilon_{y_{corr}} &= \varepsilon_{y_0}(1 - 0.0198CR) \\
nf_{u_{corr}} &= nf_{u_0}(1 - 0.0157CR) \\
E_{corr} &= E_0(1 - 0.0115CR)
\end{align*}
2.2.3.3 Buckling of Compression bars due to Corrosion

Cracking and spalling of concrete cover occurs at high corrosion levels. Smaller diameter and concrete cover of stirrups cause them to corrode before the main bars. When the corroded bars are subjected to compressive force, they may buckle because of stirrups corrosion and loss of concrete cover. Buckling force depends on axial load, un-bonded length and cross-sectional area loss of the bars. Therefore, it is extremely important to study the behavior of the reinforcing bars under compression. Bresler and Gilbert 1961, Mander et al. 1984, Mau and El-Mabsout 1989; Mau 1990, Monti and Nuti 1992, Tassios 1993, Rodriguez et al. 1994, Bayrak and Sheikh 2001; Bae et al. 2005 and many other researchers had studied the behavior of compression bars based on their material properties and geometry. For corroded compressive bars, Tassios (1993) and Rodriguez et al. (1994) suggested that when the concrete cover spalls, steel reinforcement on the compression side tends to buckle rather than yield. This is because of the loss of confinement which is provided by the concrete cover and the stirrups. With the absence of the concrete cover and stirrups, the unsupported length of the compressive steel bars increases, allowing the bars to buckle under smaller loads.
2.2.4 Cracking of Concrete due to Corrosion

Formation of rust during active corrosion and volumetric expansion of corrosion products generates tensile splitting stresses on concrete around the corroded bar. If the tensile splitting stresses exceed the maximum tensile strength of concrete, cracking occurs. Cracking starts from the interface of concrete and steel and propagates outwards. Corrosion reduces service life of concrete members once the cracks are observed on the surface of elements. Furthermore, cracking causes the concrete to lose its integrity which results in reduction of load carrying capacity of the concrete element.

Transverse tensile stresses (splitting stresses) reduce the concrete compression strength leading to generation of longitudinal cracks along the corroded bars (Vecchio and Collins, 1986). Coronelli and Gambarova (2004) proposed a model for cracked concrete in compression as follows:

\[
f_{cr} = \frac{f_c}{1 + K_\varepsilon_1/\varepsilon_{co}}
\]  

(25)

Where; \(K\) is the coefficient related to bar roughness and diameter and suggested by Cape (1999) to be considered equal to 0.1. \(\varepsilon_{co}\) is the un-cracked concrete strain corresponding to compressive strength of \(f_c\) and \(\varepsilon_1\) represents average smeared tensile strain of cracked concrete, which can be calculated as follows:

\[
\varepsilon_1 = \frac{n_{bars}w_{cr1} + w_{cr2}}{b_0}
\]  

(26)

\(b_0\) is the section width before cracking, \(n_{bars}\) is the number of corroded bars of cracked concrete in increased width of \(b_0\); \(w_{cr1}\) is the crack width formed due corrosion of longitudinal bars:
\[ w_{cr1} = 2\pi(v_{rs} - 1)P_x \]  \hspace{1cm} (27)

\( P_x \), the attack penetration factor, is the average value of corrosion penetration (bar radius reduction) in mm and \( v_{rs} \) is the ratio of volumetric expansion of corrosion products, which has been suggested by Molina et al. (1993) to be taken equal to 2.

![Figure 2-6 Constitutive law for cracked concrete in compression](image)

\( w_{cr2} \) is the crack width due to corrosion of transverse reinforcement which can be calculated from the differences of the perimeter of sound transverse reinforcement bar and the perimeter of the expansion of corrosion transverse reinforcement bar for circular sections (Asri, 2001). However, the crack width \( w_{cr2} \) can be found from the differences of the length of sound transverse reinforcement bar and the length of the expansion of corrosion transverse reinforcement bar in corroded side for rectangular sections.
Barghava et al. (2006) presented an analytical model to calculate cover cracking time. To approach this model, using the thick cylinder model, it is possible to calculate the corrosion rate at which the concrete cover cracks completely. $R_c$, the radius of cracked concrete due to corrosion according to figure can be calculated as:

$$ R_c = \sqrt{\frac{1}{\frac{f_t(1+v_{c2})}{E_{ef2}}}} $$

Where; $v_{c2}$ is the Poisson’s ratio for un-cracked concrete, $E_{ef2}$ is the effective modulus of elasticity of un-cracked concrete which depends on creep coefficient for cover concrete ($\theta$).

$$ E_{ef2} = \frac{9979.4f_{cm}^{1/3}}{1+\theta} $$

$f_{cm}$ is the 28 days cylindrical compressive strength of concrete in MPa. $f_t$ is the tensile strength of cracked concrete and proposed by CEB-FIP (1990) as:

$$ f_t = 0.302f_{cm}^{2/3} $$
Figure 2-8 Stress-strain diagram for cracked concrete cover (CEB-FIP, 1990)

\( R_o \) is the radius of concrete measuring from the center of the reinforcing bar to the exposed surface of concrete cover. \( R_{ck} \) also can be found as follows:

\[
R_{ck} = \frac{(1+\nu_{c2})u_c R_c R_o^2}{(1+\nu_{c2}) R_o^2 + (1-\nu_{c2})R_c^2}
\]  

(31)

Where; \( u_c \) is:

\[
u_c = \frac{E_{ef_1}[(1 - \nu_{c2})R_c^2 + (1 + \nu_{c2})R_o^2][2R_t R_c]}{E_{ef_1}[(1 - \nu_{c2})R_c^2 + (1 + \nu_{c2})R_o^2][(1 - \nu_{c2})R_c^2 + (1 + \nu_{c2})R_t^2] - E_{ef_2}(R_c^2 - R_t^2)(R_c^2 - R_o^2)(1 - \nu_{c1}^2)}
\]

(32)

\( R_t \) is the initial radius of the bar and \( d_c \) is the thickness of expansive corrosion product.
Figure 2-9 Concrete cover due to corrosion cracking process (Barghava et al., 2006)

In order to have the concrete cover cracked completely, it is needed to consider $R_c = R_o$ on all above equations, which results in the following equation:

$$R_t = \frac{R_c}{E_{ef2}} \cdot \frac{(1-v_{ce})R_c^2+(1+v_{ce})R_i^2}{2d_c}$$

(33)

According to CONTECVET (2001), the width of the cracks formed due to corrosion can be calculated as follows:

$$w_{cr} = 0.05 + \beta [P_x - P_{x0}] \quad w_{cr} \leq 1.0 \text{ mm}$$

(34)

Where; $w_{cr}$ is crack width in mm, $\beta$ is the bar position coefficient which is equal to 12.5 for bottom cast bars and equal to 10 for top cast bars, $P_x$ is the attack penetration in mm and $P_{x0}$ is the attack penetration corresponding to initiation of cracking. $P_{x0}$ depends on cover to diameter ratio $(c/d)$ and the splitting tensile strength $(f_{c,sp})$ in MPa:

$$P_{x0} = \left[ 83.8 + 7.4(c/d) - 22.6(f_{c,sp}) \right] \cdot 10^{-3} \quad P_{x0} \geq 0$$

(35)

In which:
\[ f_{c,sp} = 0.333 f_c^{2/3} \quad f_c \leq 50 \text{ MPa} \] (36)

Where; \( f_c \) is the characteristic cylinder strength of concrete in MPa.

As a summary, internal micro-cracking forms due to expansive corrosion products. As corrosion rate increases, external longitudinal cracks appear and finally spalling of concrete cover happens. This procedure of cracking and spalling of concrete, results in stiffness degradation of concrete. The ratio of concrete cover to bar diameter has an important role in mode of failure of spliced bars; as the ratio is reduced due to corrosion, splitting of concrete cover is more probable to occur. Radial and circumferential tensile stresses tend to split the concrete cylinder cover around the column bars. Reduction in concrete area because of cracking and spalling of concrete in a corroded column, decreases the effective cover thickness; resulting in low bond strength. Therefore, low bond strength is observed due to splitting of concrete cover because of corrosion-induced cracking (Coronelli and Gambarova, 2004). For high corrosion levels, practically there is no concrete cover left. Complete spalling of concrete cover happens when areal loss percentage is above 20% (Braverman et al., 2001).

2.2.5 Loss of Bond due to Corrosion

Bond mechanism between steel reinforcement and concrete is influenced significantly by corrosion. Bond stresses produce longitudinal, radial and circumferential tensile stresses (Lutz and Gergely, 1967). When the resulting stresses exceed the tensile strength of concrete, more cracks are formed. Steel bars have volumetric expansion due to corrosion products. This expansion generates micro-cracking in concrete which results in reduction in strength and ductility of concrete. Therefore, in a corroded concrete, formation of cracks is progressive. When cracks are formed, they propagate widely which results in quick bond deterioration and loss of
shear stress transfer between steel and concrete.

2.2.5.1 Bond Mechanism of Un-corroded RC Elements

According to ACI 408R, bond strength in general is the maximum force which can move a reinforcing bar in its longitudinal direction relative to the adjacent concrete. When reinforcing bar tends to move parallel to its length direction, bearing and friction forces act on the plane perpendicular to the bar ribs; which results in generalizing tensile stresses in both longitudinal and transverse direction of the bar. In case of insufficient concrete cover or small spacing between bars, splitting cracks occur. While providing enough concrete cover, bar spacing and transverse reinforcement lead to the longitudinal cracking and pullout failure. Bond strength mainly depends on concrete cover, bar spacing, bar casting position (which is important for beams), bar properties, steel yield strength, concrete compressive and tensile strength and amount if transverse reinforcement.

For un-corroded RC members, ACI 408R-03 presented the following equation for total bond force, which has been derived through regression analysis of test specimens:

\[
\frac{T_b}{\sqrt{f_c}} = 3\pi l_d (c_{min} + 0.4d_b) + 200A_b + \frac{\pi l_d A_{tr}}{500s} f_{yt} \tag{37}
\]

Where; \(T_b\) is the total bond force, \(l_d\) is the length of developed bar, \(A_b\) is the area of developed bar, \(c_{min}\) is the smaller of minimum concrete cover or 1/2 of the clear spacing between bars, \(n\) is the number of developed bars, \(A_{tr}\) is the area of transverse bars, \(s\) is the spacing of transverse reinforcement and \(f_{yt}\) is the yield strength of transverse reinforcement.

CEB-FIP90 presented a model for local bond-slip relation for pullout failure of a bar loaded monotonically, based on Eligehausen’s model (Eligehausen et al., 1983).
Figure 2-10 Bond-slip relationship (CEB-FIP90)

\[
\begin{align*}
\tau &= \tau_{\text{max}} \left( \frac{s}{s_1} \right)^{\alpha} & 0 \leq s \leq s_1 \\
\tau &= \tau_{\text{max}} & s_1 \leq s \leq s_2 \\
\tau &= \tau_{\text{max}} - \left( \frac{s_{\text{max}} - \tau_f}{s_3 - s_2} \right) \left( \frac{s - s_2}{s_3 - s_2} \right) & s_2 \leq s \leq s_3 \\
\tau &= \tau_f & s_3 < s
\end{align*}
\] (38)

Where; the values for slip are given in Table 2-2, at which unconfined concrete tab refers to splitting mode of failure and confined concrete tab refers to pullout failure.

<table>
<thead>
<tr>
<th>Table 2-2 Parameters to define mean bond-slip equation</th>
<th>Unconfined Concrete</th>
<th>Confined Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good bond condition</td>
<td>All other bond conditions</td>
<td>Good bond condition</td>
</tr>
<tr>
<td>s1 0.6 mm</td>
<td>0.6 mm</td>
<td>1.0 mm</td>
</tr>
<tr>
<td>s3 0.6 mm</td>
<td>0.6 mm</td>
<td>3.0 mm</td>
</tr>
<tr>
<td>s3 1.0 mm</td>
<td>2.5 mm</td>
<td>Clear rib spacing</td>
</tr>
<tr>
<td>(\alpha) 0.4</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>(\tau_{\text{max}}) (2.0 \sqrt{f'_{\text{ck}}})</td>
<td>1.0 (\sqrt{f'_{\text{ck}}})</td>
<td>2.5 (\sqrt{f'_{\text{ck}}})</td>
</tr>
<tr>
<td>(\tau_f) 0.15(\tau_{\text{max}})</td>
<td>0.15(\tau_{\text{max}})</td>
<td>0.40(\tau_{\text{max}})</td>
</tr>
</tbody>
</table>
Gan (2000) presented a model for bond-slip relationship for both splitting and pullout mode of failure. The equation for pullout failure is the same as Eligehausen’s model with slightly differences in $s_1$, $\tau_{\text{max}}$ and $\tau_f$ values.

\[
\tau_{\text{max}} = \left(20 - \frac{d}{4}\right)\left(\frac{f'_c}{30}\right)^{0.5} \quad (\text{MPa}) \tag{39}
\]

\[
\tau_f = \left(5.5 - 0.07 \frac{s}{H}\right)\left(\frac{f'_c}{27.6}\right)^{0.5} \quad (\text{MPa}) \tag{40}
\]

\[
s_1 = \left(\frac{f'_c}{30}\right)^{0.5} \tag{41}
\]

Where; $S$ and $H$ are the clear spacing and height of ribs on the bars, respectively.

For splitting mode of failure, Gong proposed the following equations for bond-slip relationship.

\[
\begin{align*}
\tau &= \tau_{\text{max}}\left(\frac{s}{s_{s1}}\right)^{\alpha} & 0 \leq s \leq s_{s1} \\
\tau &= \tau_{\text{max}} - \left(\tau_{\text{max}} - \tau_f\right)\left(\frac{s-s_{s1}}{s_{s2}-s_{s1}}\right) & s_{s1} \leq s \leq s_{s2} \\
\tau &= \tau_f & s_{s2} < s
\end{align*} \tag{42}
\]

Where;

\[
\tau_{\text{max}} = 0.748\left(\frac{f'_c}{c}\right)^{0.5} < \tau_{\text{max}} \quad (\text{MPa}) \tag{43}
\]

\[
s_{s1} = \left(\frac{f'_c}{30}\right)^{0.5} e^{\left(\frac{1}{2}\right)(\ln(\frac{\tau_{\text{max}}}{\tau_{\text{max}}}))} \quad (\text{mm}) \tag{44}
\]

\[
\tau_f = 0.15 \tau_{\text{max}} \quad (\text{MPa}) \tag{45}
\]

$s_{s2}$ is considered equal to 2mm.
Under cyclic load, the local bond-slip response is seen in Figure 2-12 (Eligehausen et al., 1983). The bond-slip behavior follows the initial part of monotonic envelope until it reaches the half of its maximum value, while permanent slip remains when unloading happens. Rigid body motion occurs on re-loading before any stress can be carried by the system. All these result in significant reduction in bond strength of the element. Balazs (1991) suggested a model for maximum sustained bond stress under reversed cyclic loading based on reduced monotonic bond-slip envelope. The reduction factor depends on the load history, the number of reversed load cycles, and the maximum previous slip (ACI 408.2R-12).
2.2.5.2 Effect of Corrosion on Bond

The volumetric extension of steel due to corrosion generates tensile stresses in concrete which effects the bond between steel and concrete. Several studies carried out on the effect of corrosion on bond indicate that as long as there is no cracking of concrete cover, the bond strength of the corroded bar increases. Beyond corrosion level of 1-4% at which concrete cover cracks, the bond strength decreases considerably and it is negligible at higher corrosion levels.
Soudki and Sherwood (2003) carried out pullout test on corroded bars embedded in concrete. They found the bond strength of the corroded reinforcing bars in concrete increased at a small level of corrosion up to 3% corrosion rate, but decreased when the degree of corrosion is beyond 3%. The bond strength of corroded specimens with 10% mass loss and with 15 and 30 mm cover is slightly lower than the corresponding un-corroded specimens.

Wang et al. (2011) also found the ultimate bond strength increases to some extent in the early stage of reinforcement corrosion, while in high corrosion rates the ultimate bond strength decreases and the failure mode changes from bond splitting to ductile mode of failure of rebar pullout.

Several empirical models have been proposed to explain bond strength of corroded reinforcing bars. Cabrera (1996) presented bond strength of corroded RC specimens, $\tau_{cor}$, based on pullout tests as:

$$\tau_{cor} = 23.478 - 1.313 C \quad (MPa)$$  \hspace{1cm} (46)

Where; $C$ is the percentage of corrosion level:

$$C = \frac{\Delta w}{w} \times 100$$  \hspace{1cm} (47)

$\Delta w$ is the average mass loss of corroded bars and $w$ is the mass of original bars.

Bond strength proposed by Lee et al. (2003) based on pullout tests of corroded RC specimens is as following:

$$\tau_{cor} = 5.21 e^{(0.0561C)} \quad (MPa)$$  \hspace{1cm} (48)
Stanish et al. (1999) presented bond strength of corroded RC members based on flexure tests as:

$$\tau_{cor} = (0.77 - 0.027 C)\sqrt{f'_c} \quad (MPa) \quad (49)$$

R, ratio of bond strength of corroded bar to bond strength of non-corroded bar was first defined by Yuan et al. (1999) as:

$$R = 1 - \left(10.544 - 1.586 \left(\frac{c}{d_b}\right)\right)C \quad (50)$$

And then Chung et al. (2004) proposed the following equation for R based on flexural tests:

$$R = 2.09 C^{-1.06} \quad \text{for} \quad C > 2.0\% \quad (51)$$

Bhargava et al. (2007) carried out experimental tests on corroded RC specimens based on both pullout and flexural tests and generalized with the following equations:

Model M-Pull (Based on pullout test)

$$\begin{cases} R = 1.0 & \text{for} \quad C \leq 1.5\% \\ R = 1.192e^{-0.117C} & \text{for} \quad C > 1.5\% \end{cases} \quad (52)$$

Model M-Flex (Based on flexural test)

$$\begin{cases} R = 1.0 & \text{for} \quad C \leq 1.5\% \\ R = 1.346e^{-0.198C} & \text{for} \quad C > 1.5\% \end{cases} \quad (53)$$

The above equations are valid for up to 10% corrosion level for model M-Flex and up to 30% corrosion level for model M-Pull. Using model M-Flex has been considered more conservative by Bhargava et al. (2007)

Confinement reduction caused by corroded transverse reinforcement plays an important role in bond degradation of main bars. Bond reduction due to corrosion is more severe when
there is no trasverse bar (Coronelli, 2002).

Yalciner et al. (2012) found that for high strength concrete, the corroded specimens, unlike the un-corroded ones, have a sudden reduction in bond strength. As the corroded specimens are brittle, at higher concrete strength levels they have more bond strength degradation at pullout test because of concrete cracking.

Fang (2006) investigate the effect of cyclic loading on corroded specimens and found that cyclic loading causes severe deterioration of bond between steel bar and concrete. Substantial reduction in bond strength has been shown in deformed corroded bars than smooth bars at first loading cycle; however, this difference was significantly lower after ten cycles. There was a considerable bond reduction in unconfined steel bars than for confined ones. At high levels of corrosion at first five cycles, bond reduction is significant, while at more number of loading cycles the effect of corrosion level decreased.

2.3 Structural Behavior of Corroded Reinforced Concrete Elements

Corrosion of reinforcing bars reduces load carrying capacity of reinforced concrete members. The first effect of corrosion on concrete members is cracking. Corrosion of several bars at one location generates splitting cracks in bars plane at initial stages, while at higher corrosion rates spalling of concrete cover occurs and the corroded bars are exposed as shown in Figure 2-14 and Figure 2-15 (Aboutaha, 2004).

As shown in Figure 2-15, corrosion reduces the cross-sectional area of reinforcing bar and even can make it fracture at high corrosion levels. The concrete cover is completely spalled off and the bond between steel bar and surrounding concrete is deteriorated (Aboutaha, 2004).
As mentioned in section 2.2.4, radial stresses are formed due to expansion of corrosion products. When radial stresses exceed the tensile strength of concrete, cracks generate between the closest exterior surface and the corroded steel bar (Aboutaha, 2004).
Figure 2-16 shows the effect of corrosion of reinforcing steel bars on the surrounding concrete (Aboutaha, 2004).

Figure 2-17 shows a corrosion-damaged rectangular column. Severely cracked and delaminated concrete cover is shown on the east elevation, where the concrete cover is still intact. However, spalling of concrete cover made the corroded bars exposed on west elevation of the column (Aboutaha, 2004).

Corrosion of steel bars in reinforced concrete structures is a major durability problem for bridges constructed in New York State (NYS). The heavy use of deicing salt compounds this problem. Given the level of deterioration in many reinforced concrete bridges in NYS, they are considered highly vulnerable to major damage during a moderate seismic event. There is an urgent need for proper guidance for evaluation of deteriorated reinforced concrete bridge components that could assist structural engineers in estimating the reserved strength of deteriorated bridges and designing cost-effective methods for retrofit. Proper evaluation and retrofit of existing deteriorated reinforced concrete bridges will limit the collapse of bridges during moderate seismic events in NYS and the surrounding states, and consequently save people’s lives. (Aboutaha et al., 2013)
Figure 2-17: Corrosion damaged rectangular concrete columns (Aboutaha, 2004)

(a) Rectangular column section.

(b) West side

(c) East side

(d) North-West side
A type of corrosion-induced cracks on circular column has been shown in Figure 2-18. “A” shows the column section with a single crack appeared near a main longitudinal rebar at the face of the column; “B” displays initial cracks with no sign of rust in the vicinity of a longitudinal rebar; and “C” demonstrates spalling of concrete cover due to an advanced state of corrosion.
corrosion, as shown in Figure 2-19 and Figure 2-20 (Aboutaha, 2004).

Figure 2-19 Corroded longitudinal and transverse bars for circular columns (Aboutaha, 2004)

Figure 2-20 Deteriorated concrete bridge pier due to corrosion of bars (Aboutaha, 2004)
Figure 2-20 and Figure 2-21 show severe deterioration of bridge concrete columns at advanced corrosion levels. Corrosion of transverse reinforcement effects confinement of the column section. Furthermore, bond deterioration between the corroded bars and the surrounding concrete decreases the load carrying capacity of the column drastically (Aboutaha, 2004).

One of the drastic causes of column failure due to corrosion is corrosion of lap-splices (Sotoud & Aboutaha, 2013). Corrosion of steel bars decreases the steel section, reduces ductility, and deteriorates the bond between the steel reinforcing bars and the surrounding concrete. Base of a bridge pier, where the main reinforcing bars are lap spliced with starter bars, is a bond-critical region in RC bridges. Contribution of different parameters such as bond stress, confinement, amount of transverse reinforcement, end bearing capacity of footing, buckling of
longitudinal bars and concrete cover thickness affect behavior of corroded lap splices. Failure of corroded lap spliced bars is controlled by limited bond stresses, as these stresses are very unlikely to reach the yield strength of the steel reinforcing bars, except for pitting corrosion cases. The higher the corrosion level, the lower the stresses that could be developed in corroded lap spliced bars. Flexural capacity of tension-controlled column section is dominated by yielding of tension bars, followed by crushing of the concrete in the compression zone. As the tensile stresses developed in corroded lap splice bars are limited by the bond stresses, flexural capacity of a column at lap-spliced section is significantly decreased (Sotoud & Aboutaha, 2014).

2.4 Experimental Studies on Corroded RC members

Behavior of corroded RC members has been investigated by many researchers. It has been proved that corrosion reduces the load carrying capacity of structural members due to reduction in cross-sectional area of steel bars, degradation of concrete due to corrosion-induced cracking and bond deterioration between corroded bar and surrounding concrete. More details are explained in the following section.

2.4.1 Studies on Behavior of Corroded RC Columns Subjected to Lateral Loads

The effect of corrosion on structural behavior of RC columns subjected to seismic loading has been studied by a few researchers.

Lee et al. (2003) experimentally investigated structural behavior of six rectangular RC columns with cross-section of 300 mm x 300 mm and height of 1100mm. Each column was reinforced with twelve D16 longitudinal bars and D10 hoops with spacing of 80mm. Electrochemical corrosion method was used to produce different levels of corrosion in hoops. The specimens were subjected to constant axial load and cyclic loading. It was found that
corrosion caused decrease in mechanical properties of rebars and spalling of concrete cover which results in reduction in confining effect of reinforcement. Mode of failure for corroded specimens was shear failing, which was caused by buckling of longitudinal reinforcement and failure of hoops.

Aquino et al. (2007) tested six circular RC columns, 500 mm in diameter and 2400 mm in height, reinforced with 12#8 longitudinal bars and #3 hoops spaced at 200 mm. External current method was used to induce corrosion in the specimens. During the test, reversed cyclic load was applied to columns and the results showed that ductility and load bearing capacity of columns are reduced due to bond deterioration caused by corrosion. Observed failure mechanism was rupture of deteriorated hoops and buckling of longitudinal bars, which resulted in shear failure of the corroded specimen.

Li et al. (2009) conducted combined lateral cyclic and constant axial loading test on fourteen RC columns to investigate the effect of combined CFRP and steel jacket retrofitting system on corroded RC columns. The specimens had cross-section of 200 mm x 200 mm and
clear height of 1500 mm; and reinforced with $4\Phi 14$ mm longitudinal bars and $\Phi 8$ mm@100 mm lateral hoops. Applying lateral cyclic load at mid-span of corroded columns, they found that by increasing the lateral load, longitudinal cracks due to corrosion developed and followed by flexural cracks. Finally, complete spalling of concrete cover due to de-bonding between concrete cover and core caused the failure of corroded columns.

Akkaya (2012) conducted lateral cyclic loading test on thirteen rectangular RC columns in three groups to study the effect of corrosion on seismic behavior of columns with plain and deformed reinforcing bars. Results of experimental tests on first and third group of columns with plain rebars and short corroded lap splices showed that bond strength in heavily corroded bars was higher than non-corroded bars, which had helped columns carry higher lateral loads. Second group of columns with deformed bars and adequate lap splice length presented very low drift capacity due to corrosion. Fracture of starter bars was the mode of failure for columns with high corrosion rate. Pitting corrosion seen at corroded lap splices was the main reason of inelastic behavior of columns subjected to lateral cyclic load.

Ma et al. (2012) carried out cyclic loading tests on thirteen circular RC columns subjected to different rates of corrosion and axial compressive loads. Circular columns with diameter of 260 mm and length of 1000 mm, having $6\Phi 16$ mm longitudinal bars and $\Phi 8$ mm spiral with pitch of 100 mm, have been corroded using external current method. With a constant axial load, reversed cyclic lateral loading was applied to the columns. They found that high corrosion levels and high axial loads led the column to fail in brittle way and cause reduction in stiffness, ductility, energy dissipation as well as poor hysteretic response. For a typical load-displacement curve of a column subjected to axial and lateral load (Figure 2-23), they determined yield load of column, $F_y$, and corresponding displacement, $\varepsilon_y$, based on assumption
of equal areas for hatched region of $S_{OAB}$ and $S_{YNB}$. They considered ultimate load of column, $F_u$, equal to 85% of maximum lateral load.

![Diagram](image)

$F_y$ and $F_u$, yield and ultimate loads of corroded column can be expressed separately in terms of $F_{y0}$ and $F_{u0}$, yield and ultimate load of un-corroded column and $\rho$, corrosion rate, based on regression of test data:

$$F_y = F_{y0}(1 - 0.885\rho) \quad (54)$$

$$F_u = F_{u0}(1 - 0.878\rho) \quad (55)$$

Where; $\rho$ is the average corrosion level, in terms of $G_0$, the initial weight of steel before corrosion and $G$, the final weight of steel after corrosion.

$$\rho = \frac{(G_0 - G)}{G_0} \times 100\% \quad (56)$$

Meda et al. (2014) conducted combined lateral cyclic and constant axial loading test on two RC columns to investigate the effect of corrosion on corroded RC columns. The specimens had a cross-section of 300 mm x 300 mm and a clear height of 1500 mm; and were reinforced...
with 4Φ16 mm longitudinal bars and #8 mm@300 mm lateral hoops. Applying lateral cyclic load at the end of un-corroded columns, they found that by increasing the lateral load, flexural cracks were developing. After complete yielding of longitudinal bars and large deformation of compression bars due to buckling, the column experienced the maximum lateral load and then due to concrete crushing and cover spalling the test had been stopped. Same loading on corroded columns showed that by increasing the lateral load, longitudinal cracks due to corrosion developed and followed by flexural cracks. Finally, complete spalling of concrete cover due to de-bonding between concrete cover and core, crushing of concrete and buckling of corroded bars caused the failure of corroded columns. Lateral capacity of the corroded column decreased by 27% in comparison with the lateral capacity of un-corroded specimen.

Figure 2-24 Load–deformation curves of un-corroded vs. corroded column (Meda et al., 2014)
2.4.2 Studies on Lateral Behavior of RC Corroded Elements

The lateral behavior of corroded RC elements has not been studied widely. Most of the studies on corroded columns are those subjected to just axial load or eccentric axial load which produces bending moment. Very few studies has been done for corroded columns subjected to combined axial and lateral loads. Few studies investigated the shear response of RC beams which is similar to columns, but without considering axial loads.

In a comprehensive study on corroded RC beams by Rudriguez et al. (1997), it was found that while a non-corroded beam fails by bending, deteriorated beam fails by shear. Beam deterioration can change the failure mechanism from ductile to brittle. The ultimate flexural and shear strength of beams can be calculated using the current design codes with consideration of reduced concrete and steel bar section. Such approach would produce a very rough estimate of strength, and only true for limited deterioration.

Maaddawy et al. (2005) carried out vertical loading on beams to investigate lateral performance of corroded beams. The specimens with 152 mm in width and 254 mm in height had 2\#5 longitudinal bars at bottom, 2Ø8 bars at top, Ø8@80 mm stirrups in shear span and Ø8@333.33 mm stirrups in the constant moment region. Accelerated corrosion imposed to longitudinal bars of all specimens in two groups; (beams with no load, and beams with sustained load). Four-point bending loading is applied to the system. The results of the test showed that the beams which have been corroded under sustained load, had more mass loss due to corrosion and the crack width due to corrosion was higher in this group of beams. However, strength reduction due to corrosion for high levels of corrosion were the same for both groups of beams.

Xue and Seki (2010) investigated the effect of corroded longitudinal bars on shear
behavior of RC beams. They conducted experimental tests on five series of beams with different
corrosion rate and shear-span-to-effective-depth-ratio (a/d). The specimens had 120 mm in
width, 240 mm in height and 2D9 mm tension steel bars without any stirrups. They considered
five a/d ratios of 1.5, 2.0, 2.6, 3.5 and 4 to investigate the effect of shear span on failure mode of
beams. Electrochemical method had been used to induce corrosion to bars. The beams were
loaded at mid-span considering simply supported condition. Corrosion of longitudinal bars
results in smaller cross-sectional area and more important, reduction in bond strength and
generation of cracks in concrete. They found that deterioration of bond strength between
contact and steel bars reduces the stiffness of beam and can change failure mechanism of beam.
Transition of load carrying mechanism changes load-bearing capacity of beams which may not
be as expected. The analytical study on same situation of beams lead Xue and Seki to propose an
equation for shear capacity of corroded RC beam as:

$$V_{u-\text{eval}} = \{1 + \left[0.13 - 0.04 \left(\frac{a}{d}\right)\right] \cdot C\} \cdot V_u$$  (57)

Where; $V_u$ is shear capacity of sound specimen and $C$ is average mass loss to describe the
corrosion rate.

$$C = \frac{\Delta w}{w} \times 100$$  (58)

Where; $\Delta w$ is the average mass loss of corroded bars and $w$ is the mass of original bars.

Shihata (2011) tested eighteen reinforced concrete beams to examine the effect of CFRP
sheets on improving the bond strength of corroded lap splices. The beams consisted of two
tension bars which spliced at mid-span. Spliced region had no transverse reinforcement and was
subjected to constant bending moment. Concrete-cover to bar diameter and corrosion level were
the parameters investigated in this research. Results of experiments showed brittle failure of longitudinal bars in spliced region due to de-bonding and splitting of concrete cover. Maximum strain of bars never reached the yielding strain due to short length of lap splice. Furthermore, increase in corrosion level decrease the bond strength in longitudinal bars.

Juarez et al. (2011) tested two groups of corroded beams with different rates of corrosion and different spacing between stirrups. The simply supported beam with clear span of 1800 mm and width of 200 mm and height of 350 mm had 5#16 longitudinal bars and #8 stirrups with a spacing of 150 mm and 200 mm in two separate groups. Longitudinal bars were covered by a resin-based epoxy anticorrosive paint. Chloride-induced corrosion was imposed to specimens. Nominal shear capacity of beams calculated using ACI318-08 equations; considering the reduced (critical) diameter of corroded stirrups.

They applied concentrated loads to the beam to find the ultimate shear strength. Comparison between the nominal and ultimate shear strength of corroded RC beams indicated that using the critical diameter, and not the average one, gives a conservative and reliable prediction of ultimate shear strength.

Ou et al. (2012) carried out cyclic loading on five large-scale beams to investigate cyclic performance of corroded beams. The specimens 30 cm in width and 50 cm in height had 5#5 longitudinal bars at top and bottom and #3@10 cm stirrups. The cantilever beams were designed according to ACI 318 and transverse reinforcement was designed in a way that flexural failure mode was controlling. Accelerated corrosion was imposed to all specimens except the sound one. Cyclic loading was applied to the free end of the cantilever and the results showed that because of rupture of transverse reinforcement due to corrosion, longitudinal bars buckled and
failure mode switched from flexural failure mode to flexural-shear one.

Figure 2-25 Crack pattern, (a) to (e): increase in corrosion rate from, (f) close view of fracture of transverse reinforcement for (e) (Ou et al., 2012)

2.5 Finite Element Modeling of RC Columns

Modeling of structures gives a reasonable estimation of performance of infrastructure systems. The nonlinear response of RC structures can be computed using the finite element method. It has been approved by several researchers that finite element method is a convenient tool to assess the behavior of reinforced concrete members.

Xiaoming and Hongqiang (2012) carried out finite element (FE) study on load carrying
capacity of corroded RC beam based on the bond-slip between the steel bars and concrete. Using ANSYS, they modeled several RC simply supported beams. Different corrosion rates have been investigated. In FE models, to simulate concrete, element of Solid65 was used. Link8 has been used to model bars and Combin39 have taken to simulate the bond between bars and concrete. Corrosion rate affected the cross-sectional area and yielding strength of bars and bond force between bars and concrete. Based on results, at higher corrosion rates the stiffness of corroded beams decreases while the slip between bars and concrete increases and mode of failure of beams changed from ductile to brittle failure. The speed of reduction in load carrying capacity of corroded RC beam is significantly high at the corrosion levels in the range of 4%-7%.

Lettow et al. made a finite element model emphasized on bond model which accounts for the impact of the reinforcement strains, the stress of the surrounding concrete and the cyclic loading history on bond strength. They assumed the connection of bars to concrete in transverse direction is perfect, so the bond elements have been used just in longitudinal direction to be able to simulate slip. The finite-length and zero-width descript bond element is a two-node finite element connects a bar finite element with a three-dimensional concrete solid finite element. There is a good agreement between finite element models of tension member and pullout tests considering short and long embedment length. Also the bond finite element has a fine prediction of bond stresses transfer from reinforcement into concrete, especially for large yielding strains.

Potisuk et al. (2011) conducted a finite element model in ANSYS to develop the different contributions of corrosion damage parameters to structural behavior of experimental RC beams with shear-dominated behavior. The parameters investigated were concrete cover spalling, uniform and localized cross-sectional loss of stirrups due to uniform and pitting corrosion and debonding of corroded stirrups from the concrete. Considering both individual and combined
damages in FE models, the numerical analysis were compared to experimental results which indicated a good agreement.

Berra et al. (2005) studied the effect of corrosion on bond degradation making a finite element model on ABAQUS. Three dimensional axi-symmetric elements with a corroded steel bar and the surrounding concrete in different confinement levels with varying transverse steel percentage and different arrangement of them were modeled. A model was proposed based on corrosion product expansion which results in cover cracking.

Using DIANA, Lundgren (2005, 2007) performed a finite element analysis with solid elements for concrete and steel bars. To model band, the interface elements were used to simulate relative displacement of steel bar to concrete. Both friction bond model (Columbus Friction) and corrosion model in terms of mechanical behavior and volume of corrosion products were considered. For different corrosion levels and both ribbed and smooth bars, the finite element model had a reasonable agreement with experimental results.

Castellani and Coronelli (1999) modeled a RC beam using plane elements with equal thickness of beam for concrete, truss element for reinforcing bar and link elements to simulate bond between bars and concrete. They used modified CEB-FIP model for bond-slip of corroded bars and investigated different parameters as bond deterioration, bar area loss, cracking of concrete and spalling of cover to have an estimation of response of corroded concrete element. Later on, Coronelli and Gambarova (2004) improved the bond-slip model and analyzed several corroded beams with a good agreement with experimental data.

Horrigmoe and Hansen (2004) conducted a numerical analysis on corroded RC beams with only tension bars. Bond-slip law specified by Torlen and Horrigmoe (1998) was used to
simulate the bond reduction between corroded bars and concrete in ANSYS. In addition to bond modeling, the uniform cross-sectional loss due to corrosion was considered. Excellent agreement with experimental results has been achieved.

Sand (2001) conducted a finite element analysis on ANSYS, modeling a RC beam with corroded tensile reinforcement (both uniform and pitting corrosion) and corroded spliced tensile reinforcement (only uniform corrosion). In order to simulate the service life cycle of corroded beam, a sequence of phases with serviceability load, corrosion, partial unloading, repair loading and failure loading were studied. Different corrosion levels as well as different exposed length of corroded bars were investigated. The finite element model had a realistic estimation of response of corroded beam tests.

Vu et al. (2016) performed a finite element analysis on DIANA, modeling 240 corroded RC columns subjected to seismic loading. Columns had the cross-section of 350 mm x 350 mm and aspect ratios of 4, 3 and 2. Each column was reinforced with 8T16 longitudinal bars and two D6 hoops with different spacing. Concrete compressive strength of 30 and 40MPa, axial load ratios of 0.1, 0.2, 0.3 and 0.4, and corrosion levels of 0, 5, 10, 20 and 30 percent were studied. They studied effect of different parameters on seismic response of corroded columns and proposed equations to predict lateral strength and lateral drift of corroded bars. More explanation on equations is given in section 2.6. The FE results showed that increase of aspect ratio results in reduction in the lateral load resistance of corroded RC columns; however, corroded RC columns have less lateral capacity when they are subjected to lower axial load, less compressive concrete strength and low stirrup amount. The effect of stirrup amount is noticeable in higher corrosion levels.
2.6 Analytical Models to Estimate Lateral Capacity of Corroded RC Members

As mentioned before, there are a small number of analytical models developed for corroded RC members. Webster (2000) presented an equation for shear strength of corroded RC beams with no shear reinforcement. This equation is based on regression of test results on beams with different c/D ratios and depends on

\[ V_{c,corr} = \left( \frac{A_{seff}}{A_s} \right)^{1/3} V_c \]  \hspace{1cm} (59)

Where;

\[ \frac{A_{seff}}{A_s} = 0.8 \left[ \frac{c}{D} \right]^{0.3} \left[ \frac{\tau_{corr}}{\tau_{8110}} \right]^{0.3} \] \hspace{1cm} (60)

Where; \( A_{seff} \) is the effective area of corroded tension bars and \( A_s \) is the nominal area of un-corroded tension bars, \( c/D \) is the cover to tension bar diameter ratio, \( \tau_{corr} \) is the bond strength of corroded tension bars and \( \tau_{8110} \) is the bond strength of un-corroded tension bars in advance with BS 8110. In the presence of shear reinforcement the term \( V_s \) according to BS 8110 is added to \( V_{c,corr} \).

The analytical study on the same beams, led Xue and Seki (2010) to propose an equation for shear capacity of RC beam with corroded longitudinal bars as:

\[ V_{u-\text{eval}} = \{1 + \left[ 0.13 - 0.04 \left( \frac{a}{d} \right) \right] \cdot C \} \cdot V_u \] \hspace{1cm} (61)

Where; \( V_u \) is shear capacity of sound specimen and \( C \) is average mass loss to describe the corrosion rate.

As well as Webster (2000), Juarez et al. (2011) also considered the contribution of
corrosion on shear strength of concrete only. \( \lambda \) is the corrosion factor related to unit weight of concrete in the following equation:

\[
V_n = 0.17\lambda\sqrt{f'_c} b_w d + \frac{A_{ov} f_y d}{s}
\]  

(62)

According to ACI 318-11, nominal shear capacity of RC columns shall be defined as:

\[
V_n = V_c + V_s
\]  

(63)

\( V_c \) is nominal shear strength provided by concrete, which can be calculated using the following equations for concrete members subjected to axial compression loads.

\[
V_c = (1.9\lambda\sqrt{f'_c} + 2500\rho_w \frac{V_{ud}}{M_m} b_w d \leq 3.5\lambda\sqrt{f'_c} b_w d \sqrt{1 + \frac{N_u}{500A_g}}
\]  

(64)

Where:

\[
M_m = M_u - N_u \frac{4h - d}{8}
\]  

(65)

For corroded section, Higgins et al. (2003) suggested using \( b_{eff} \) instead of \( b \) due to decrease in cross section of concrete.

\[
\begin{align*}
  b_{eff} &= b - 2(c_v + d_s) + \frac{s}{5.5} & \text{if } s \leq 5.5c_v \\
  b_{eff} &= b - \frac{5.5}{s}(c_v + d_s)^2 & \text{if } s > 5.5c_v
\end{align*}
\]  

(66)

where \( b \) is the original undamaged section width (in), \( c_v \) is the concrete cover (in), \( d_s \) is the stirrup diameter (in), and \( s \) is the stirrup spacing (in).
$V_S$ is the nominal shear strength provided by shear reinforcement and for corroded RC section, $A_v$, the cross-sectional area of corroded stirrups can be calculated considering corrosion rate.

Strut-and-tie modeling of corroded beams by consideration of average cross-sectional loss and reduced effective beam width by Higgins et al. (2010) verified STM for corroded RC members. Azam and Soudki (2012) studied the behavior of deep beams with corroded longitudinal bars and found that failure mode can be splitting of strut or yielding of longitudinal bars. The following equations for STM of corroded beams showed a good agreement with experimental results.

$$
V = \left(0.6 f'_c \right) w_c \cdot b \cdot \sin \theta \quad \text{Failure due to splitting of strut}
$$

$$
V = A_s (1 - C) F_y \cdot t \sin \theta \quad \text{Failure due to yielding of longitudinal bars}
$$

where; $w_c$ is the width of strut, $b$ is the width of beam, $C$ is the corrosion rate and $\theta$ is the angle between the diagonal strut and longitudinal bars.

Khan et al. (2013) used Tang and Tan’s model for their STM to calculate the shear resistance of corroded beams. They just considered reduced cross-sectional area of corroded bars and modified $A_s$ and $A_w$ in equation (72).

Vu et al. (2016) performed a finite element analysis on DIANA, modeling corroded RC
columns subjected to seismic loading. They studied the effect of different parameters on seismic response of corroded columns and proposed equations to predict lateral strength and lateral drift of corroded bars:

\[
V_{nc} = \left(\frac{1}{1+1.59X_{corr}^{100}}\right)^{0.7A_{f,y}d} + \frac{0.35}{0.5} \sqrt{\frac{f' c}{N}} 0.8A_g \right) (68)
\]

\[
DR_{ultc} = \left(\frac{1}{1+1.88X_{corr}^{100}}\right)^{(0.45 + 0.21\rho_y)}(1.66 + \frac{0.26}{f' cA_g}) (1.49 + 0.12 \frac{a}{d}) (69)
\]

Where, \(V_{nc}\) and \(DR_{ultc}\) are the lateral load resistance and the ultimate drift capacity of the corroded RC column, respectively and \(X_{corr}\) is the corrosion level. The effects of corrosion level, aspect ratio, axial force ratio, concrete compressive strength, and transverse reinforcement ratio have been investigated.

### 2.7 Summary and Conclusion

Corrosion of steel reinforcement in concrete structures results in three main losses:

- Losses in the mechanical performance of reinforcing bars due to the losses in their cross-sectional area and ductility,
- Losses in the effective cross-sectional area of concrete due to cracking in the cover concrete,
- Losses in the bond performance of concrete with reinforcements.

When corrosion occurs uniformly, considering area loss percentage of corroded bars is preferred instead of weight loss percentage. Obviously, for corroded columns, de-bonding is inevitable in high corrosion levels. For severely corroded RC members, concrete cover is totally spalled off.
Table 2-3 shows the relation between corrosion rate of longitudinal and transverse reinforcement according to data provided by Ou et al. (2012). When the corrosion rate in longitudinal bars is more than 10%, practically there is no transverse bar and the contribution of stirrups can be ignored.

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Corrosion Rate (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitudinal Bars</td>
<td>1.38 1.8 2.19 3.37 ........ 9</td>
</tr>
<tr>
<td>Transverse Bars</td>
<td>1.7 3.08 4.08 8.03 ........ Fracture</td>
</tr>
</tbody>
</table>

Considering all the above-mentioned facts, in a severely uniform corroded RC column there is no concrete cover and transverse bars at the corroded side. There is no bond between the corroded bar and surrounded concrete; therefore, the corroded bar in compression acts as a bare bar at which the compression stress is limited by buckling force.

There are no studies on severely corroded columns. Most of the analytical models proposed for corroded RC elements are for beams with no axial load and mostly are based on regression analysis of experimental data. De-bonding of corroded bars and its consequences such as buckling and nonlinear strain distribution in the corroded column section makes it impossible to follow the regular analysis of sections. On the other hand, the proposed equations for shear strength, even for un-corroded RC elements is based on empirical tests. Therefore, regression analysis of data obtained from experimental tests or finite element analysis in proposing a practical model to calculate the lateral capacity of severely corroded RC columns is inevitable.
3 FINITE ELEMENT MODELING

3.1 Introduction

Modeling of structures gives a reasonable estimation of performance of infrastructure systems. The nonlinear response of RC structures can be computed using the finite element method. It has been demonstrated by several researchers that finite element method is a convenient tool to assess the behavior of reinforced concrete members.

Having eight-node solid elements to model concrete with tensile cracking and compressive crushing ability, and 3D beam elements to model reinforcing bars with elastic-plastic property, ABAQUS has the capability to analyze RC members non-linearly. A finite element model has been developed using ABAQUS (ver. 6.14) to simulate response of corroded reinforced concrete columns. Then the results based on simulated model, have been verified by existing experimental data.

3.2 Element Types and Material Properties

All corroded reinforced concrete members are modeled with solid elements representing concrete, and beam elements representing reinforcing bars. As corrosion level in all studied cases is above 10%, no bond between bars and surrounding concrete has been defined. It means there is no transverse restriction in the direction of axial axis of bars. Therefore, the connection between corroded bars and concrete has been limited just in above-mentioned direction. It is important to mention that in studied experimental cases, there is still concrete cover while the corrosion rate is below 25%. However when corrosion rate is above 25%, there is no concrete cover left.
3.2.1 Concrete

Continuum (Solid) Element is an 8-node linear brick element type which can model concrete members with or without rebar. This solid element has the ability of cracking under tension and crushing in compression. These properties in addition to rebar capability make this element ideal for concrete modeling. As shown in Figure 3-1, the continuum element has eight nodes. Translations in the x, y, and z directions are the three degrees of freedom for each node.

![Figure 3-1 Continuum (solid) element geometry (ABAQUS 6.14)](image)

The concrete damage plasticity model in ABAQUS with abilities of cracking and crushing gives the capability of acting like a nonlinear material. The concrete can crack in three orthogonal directions; it can crush and has plastic deformation.

In order to model concrete, linear isotropic and multi-linear isotropic properties must be input. The linear isotropic can be defined as follows (AASHTO LRFD, 2012):

\[
E_c = 33,000 \ (w_c)_{1.5} \sqrt{f_c}
\]  

(70)

Where; \(E_c\) is modulus of elasticity of concrete (ksi), \(w_c\) is unit weight of concrete and is
considered as equal to 0.150 k/cf for normal weight concrete and \( f'_{c} \) is the specified compressive strength of concrete (ksi). The input value for modulus of elasticity in ABAQUS should be equal to the initial slope of stress-strain diagram of concrete. For multi-linear isotropic properties, stress-strain relation of concrete were defined based on modified Hognestad model for unconfined concrete. The model developed by Saatcioglu and Razvi (1992) was used for confined concrete.

### Table 3-1 Concrete models in compression

<table>
<thead>
<tr>
<th>Modified Hognestad Model for Unconfined Concrete:</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{co} = f^{'}<em>{co} \left[ 2 \left( \frac{\varepsilon</em>{co}}{\varepsilon_{01}} \right) - \left( \frac{\varepsilon_{co}}{\varepsilon_{01}} \right)^2 \right] )</td>
</tr>
<tr>
<td>( \varepsilon_{01} = 1.8 \frac{f^{'}_{co}}{E} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Saatcioglu and Razvi’s Model for Confined Concrete:</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{c} = f^{'}<em>{cc} \left[ 2 \left( \frac{\varepsilon</em>{c}}{\varepsilon_{1}} \right) - \left( \frac{\varepsilon_{c}}{\varepsilon_{1}} \right)^2 \right]^{1/(1+2K)} ) ( \leq f^{'}_{cc} )</td>
</tr>
<tr>
<td>( f^{'}<em>{cc} = f^{'}</em>{co} + k_{1} \cdot f_{le} )</td>
</tr>
<tr>
<td>( k_{1} = 6.7 \left( f_{le} \right)^{-0.17} )</td>
</tr>
<tr>
<td>( f_{le} = k_{2} \cdot f_{l} )</td>
</tr>
<tr>
<td>( k_{2} = 0.26 \sqrt{\frac{b_{c}}{s}} \cdot \left( \frac{b_{c}}{s_{l}} \right) \cdot \left( \frac{1}{f_{l}} \right) ) ( \leq 1.0 )</td>
</tr>
<tr>
<td>( f_{l} = \sum A_{s} \cdot f_{yl} \cdot \sin \alpha ) ( \frac{1}{s \cdot b_{c}} )</td>
</tr>
<tr>
<td>( \varepsilon_{1} = \varepsilon_{01} \left( 1 + 5K \right) )</td>
</tr>
</tbody>
</table>

\( f_{c} = \) Stress in concrete (in MPa);
\( f^{'}_{cc} = \) Confined concrete strength in member (in MPa);
\( f^{'}_{co} = \) Unconfined concrete strength in member (in MPa);
\( E'_{c} = \) Modulus of elasticity of concrete (in MPa);
\( f_{le} = \) Equivalent lateral pressure that produces the same effect as uniformly applied pressure;
\( f_{l} = \) Average lateral confinement pressure (in MPa);
\( A_{s} = \) Area of one leg of transverse reinforcement (in mm²);
\( f_{yl} = \) Yield strength of transverse reinforcement (in MPa);
\( b_{c} = \) Core dimension measured center-to-center of perimeter hoop (in mm);
\( s = \) Spacing of transverse reinforcement in longitudinal direction (in mm);
\( S_{l} = \) Spacing of transverse reinforcement, laterally supported by the corner of a hoop or the hook of a cross tie (in mm);
\( \varepsilon_{c} = \) Concrete strain;
\( \varepsilon_{1} = \) Strain corresponding to peak stress of confined concrete;
\( \varepsilon_{01} = \) Strain corresponding to peak stress of...
For Rectangular columns:

\[ f_{le} = \frac{f_{lex} \cdot b_{cx} + f_{ley} \cdot b_{cy}}{b_{cx} + b_{cy}} \]

unconfined concrete;

\( \varepsilon_{85} = \text{Strain corresponding to 85 \% of peak stress of confined concrete on the descending branch;} \)

\( \varepsilon_{085} = \text{Strain corresponding to 85 \% of peak stress of unconfined concrete on the descending branch;} \)

\( K, k_1, k_2 = \text{Coefficients;} \)

\( \rho = \text{Reinforcement ratio.} \)

\( b_{cx}, b_{cy} = \text{Core dimensions in x and y directions, respectively (in mm);} \)

\( f_{lex}, f_{ley} = \text{Equivalent lateral pressures perpendicular to } b_{cx} \text{ and } b_{cy}, \text{respectively (in MPa);} \)

Figure 3-2 shows the defined concrete model in ABAQUS. The first point on the graph, the point corresponding to \( f_c = 30\% f'_{c} \), defines the end point of linear branch of stress-strain diagram up to which Hook’s law applies (Kachlakev et al., 2001). The remaining points are calculated based on the equations defined on Table 3-1 until the ultimate strain. It is important to note that the initial slope of stress-strain diagram is almost equal to the modulus of elasticity which is calculated based on the above-mentioned equations.

Figure 3-2 Concrete model in compression for \( f'_{co} = 4000 \text{ psi} \)
Poisson’s ratio is considered to be 0.2. ABAQUS assumes a linear stress-strain relationship for concrete in tension until the uniaxial cracking stress (modulus of rupture) is reached. Beyond the uniaxial cracking stress, softening of concrete occurs due to macroscopical development of cracks. Belarbi and Hsu’s model (1994) is used to model concrete tensile behavior.

<table>
<thead>
<tr>
<th>Table 3-2 Concrete models in tension</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Hsu’s Model</strong></td>
</tr>
<tr>
<td>$f_t = E_c \varepsilon_t$</td>
</tr>
<tr>
<td>$f_t = f_r \left( \frac{\varepsilon_t}{\varepsilon_{cr}} \right)^{0.4}$</td>
</tr>
<tr>
<td>$f_r = \text{Modulus of rupture (in MPa)}$;</td>
</tr>
<tr>
<td>$f_t = \text{Stress in concrete (in MPa)}$;</td>
</tr>
<tr>
<td>$\varepsilon_t = \text{Strain in concrete (in MPa)}$;</td>
</tr>
<tr>
<td>$\varepsilon_{cr} = \text{Corresponding strain to } f_r \text{ (in MPa)}$;</td>
</tr>
</tbody>
</table>

Modulus of rupture for specified concrete strengths up to 15.0 ksi can be defined as following (AASHTO LRFD 2012):

$$f_r = 0.24 \sqrt{f'_c}$$

(71)

Where; $f'_c$ is specified compressive strength of concrete in ksi.

![Figure 3-3 Concrete model in tension for $f'_c = 4000 \text{ psi}$](image-url)
The modulus of rupture and the concrete compressive strength represent the uniaxial cracking stress and the uniaxial crushing stress, respectively. To avoid time errors in ABAQUS material model, the Hsu’s model needs to be softened immediately after the onset of cracking. Figure 3-3 shows the concrete model in tension, the original model and the modified one.

3.2.2 Reinforcing Steel

Beam Element is the 3D uniaxial tension-compression element type which can model reinforcing steel members. This spar element is defined by two nodes as shown in Figure 3-4; each node has six degrees of freedom, three translations in and three rotations about x, y, and z direction. Beam element has the capabilities of bending, plasticity, creep, rotation, large deflection, and large strain (ABAQUS 6.14). Sound steel reinforcement is modeled by 3D Beam element as well as corroded steel reinforcement in this study.

The real constant defined for steel reinforcement is the cross section area of bar, which can be varied for different bar sizes. Modulus of elasticity is assumed equal to 29000 ksi, unless another value has been mentioned in experimental data by others. Poisson’s ratio is considered to
be 0.3. The stress-strain relation with strain hardening (Akkari and Duan (2000), Chai et al. (1990)) is defined as in Table 3-3 for un-corroded reinforcing steel members.

\[
\begin{align*}
\varepsilon_{sy} & = \begin{cases} 
60 \varepsilon_s & \text{for Grade 40} \\
5 \varepsilon_s & \text{for Grade 60}
\end{cases} \\
\varepsilon_{sh} & = \begin{cases} 
14 \varepsilon_y \\
5 \varepsilon_y
\end{cases} \\
\varepsilon_{su} & = \begin{cases} 
0.14 + \varepsilon_{sh} \\
0.12
\end{cases}
\end{align*}
\]

Table 3-3 Reinforcing Steel Model for Un-corroded bars

<table>
<thead>
<tr>
<th>Condition</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 ≤ (\varepsilon_s) ≤ (\varepsilon_y)</td>
<td>(f_y)</td>
</tr>
<tr>
<td>(\varepsilon_{sy}) &lt; (\varepsilon_s) ≤ (\varepsilon_{sy})</td>
<td>(f_y (m(\varepsilon_s - \varepsilon_{sy}) + 2 + (\varepsilon_s - \varepsilon_{sy}) (60 - m) / 2(30r + 1)^2))</td>
</tr>
<tr>
<td>(\varepsilon_{sy}) &lt; (\varepsilon_s) ≤ (\varepsilon_{su})</td>
<td>(f_y (m(\varepsilon_s - \varepsilon_{sy}) + 2 + (\varepsilon_s - \varepsilon_{sy}) (60 - m) / 2(30r + 1)^2))</td>
</tr>
</tbody>
</table>

For corroded reinforcing steel bars, different proposed properties in 2.2.3 were examined to check the best agreement with experimental data and the following equations have been verified to use. Cross sectional area \(A_{corr}\), yielding stress \(f_{y_{corr}}\) and ultimate strain \(\varepsilon_{su_{corr}}\) of corroded reinforcing steel bar are calculated based on the initial cross sectional area \(A_0\), yielding stress \(f_{y_0}\) and ultimate strain \(\varepsilon_{su_0}\) of un-corroded bar considering the area loss (corrosion level) of corroded bars \((CR)\) (Du et al., 2005):

\[
\begin{align*}
A_{corr} & = A_0 (1 - 0.01 CR) \\
f_{y_{corr}} & = f_{y_0} (1 - 0.005 CR) \\
\varepsilon_{su_{corr}} & = \varepsilon_{su_0} (1 - 0.005 CR)
\end{align*}
\]

Figure 3-5 shows the tensile stress-strain relation with strain hardening for Grade60 sound steel bar and a corroded Grade60 bar with 25% corrosion level.
As there is no interaction between corroded bars and concrete, the equations of the Structural Stability Research Council (SSRC) in its third edition of the Guide (Chen and Lui 1987), has been used in order to compute the buckling stress of steel reinforcement bars in the compression zone. The adopted equations account for the critical buckling stress of solid circular columns as in Table 3-4.

![Figure 3-5 Tensile stress-strain diagram of un-corroded and corroded steel (Grade 60)](image)

As there is no interaction between corroded bars and concrete, the equations of the Structural Stability Research Council (SSRC) in its third edition of the Guide (Chen and Lui 1987), has been used in order to compute the buckling stress of steel reinforcement bars in the compression zone. The adopted equations account for the critical buckling stress of solid circular columns as in Table 3-4.

<table>
<thead>
<tr>
<th>Table 3-4 Compressive Reinforcing Steel Model for Corroded Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_b = \begin{cases}</td>
</tr>
<tr>
<td>1 \text{ (Yield level)} &amp; (0 \leq \lambda_c \leq 0.15) \cr</td>
</tr>
<tr>
<td>1.035 - 0.202\lambda_c - 0.222\lambda_c^2 &amp; (0.15 \leq \lambda_c \leq 1.0) \cr</td>
</tr>
<tr>
<td>-0.111 + 0.636\lambda_c^{-1} + 0.087\lambda_c^{-2} &amp; (1.0 \leq \lambda_c \leq 2.0) \cr</td>
</tr>
<tr>
<td>0.009 + 0.877\lambda_c^{-2} &amp; (2.0 \leq \lambda_c \leq 3.6) \cr</td>
</tr>
<tr>
<td>\lambda_c^{-2}(\text{Euler buckling)} &amp; (\lambda_c &gt; 3.6) \cr</td>
</tr>
<tr>
<td>\end{cases} $</td>
</tr>
</tbody>
</table>

\[
\lambda_c = \frac{K.L}{r.\pi} \frac{f_y}{\sqrt{E_s}}
\]

$f_b$ = Critical buckling stress in steel; $E_s$ = Modulus of elasticity of steel; $f_y$ = Yield stress; $K$ = Effective length factor; $L$ = Laterally unbraced length of member; $r$ = Radius of gyration about the axis of buckling.
Severely corroded bars are assumed to act as pinned ends single bars with effective length factor of 1. All material properties of corroded bars have been adjusted based on corrosion rate. For all studied columns in this research, buckling is in elastic range which means if the force is removed, buckled bars could straighten up. According to Figure 3-7, when a bar buckles in elastic range, adding more axial load does not affect its load carrying capacity. It means the ultimate force a buckled bar could carry is considered equal to critical buckling force calculated according to Table 3-4.

Figure 3-6 Buckling stress diagram of corroded steel (Grade 60)

Figure 3-7 Large displacement load-deflection behavior of a pinned-ended column (Chen & Lui, 1987)
3.2.3 Rigid Plate

In order to avoid stress concentration at locations on which the loads are applied, a rigid steel plate is modeled. Continuum element type with elastic property is used to model the rigid plate. A very high modulus of elasticity and 0.3 Poisson’s ratio has assumed for the element to present a rigid plate transferring the loads to the reinforced concrete uniformly.

3.2.4 Contact Element

In cases with corrosion level less than 25%, the concrete cover is simulated as it is and a contact element between concrete cover and core has been defined to simulate the cracks between cover and core due to corrosion. For the corroded reinforcing bars, as well, there is no bond between steel and concrete, so the contact element has been defined around the corroded bars. The property of contact element in normal direction is defined as hard contact and in tangential directions it is assigned as frictionless.

3.3 Loading

All the columns in this research, including experimental test columns studied by others and the current study’s FE models are modeled as cantilevers in ABAQUS. Column bases are defined as fixed supports; then all degrees of freedom at all nodes of base elevation are restricted as shown at Figure 3-8. Self-weight of columns is considered in models. The first step of analysis is applying axial load to the rigid plate at top of the column. Next step is applying monotonic lateral load; at which lateral displacement is applied to the rigid plate.
3.4 Nonlinear Static Analysis

Both two steps of loading, axial force and lateral displacement, are static loads and divided into a series of load increments as sub-steps. The stiffness matrix of the model is updated after completion of nonlinear analysis at each sub-step. Newton-Raphson equilibrium iteration is used to provide convergence at the end of each sub-step and adjust the stiffness matrix of model before proceeding to the next sub-step. The convergence criteria in this study is defined to be based on force control and displacement control.

3.5 Validation of Experimental Data

In order to investigate the response of corroded RC columns subjected to lateral loads, it is necessary to validate the FE model against existing experimental test data. Existing experimental data includes the un-corroded and corroded columns subjected to axial and lateral loads, and corroded beams subjected to shear forces. All the specimens were modeled according
to data provided on related papers. The stress-strain relationships for both steel and concrete have been considered as discussed in section 3.2, if they were not available.

3.5.1 Modeling of Corroded Elements

Behavior of corroded RC members has been investigated by some researchers. Just a few of this studies are on corroded columns and among those columns, two experimental tests (Gong, 2009 and Meda et al., 2014) have carried out on corroded columns with rectangular section and no lap-splices. In addition to corroded test columns, corroded test beams (representing columns without axial load) by Maaddawy et al. (2005) and Ou et al. (2012) have been modeled, too.

3.5.1.1 Maaddawy et al. (2005)

Maaddawy et al. (2005) carried out vertical loading on beams to investigate lateral performance of corroded beams. The specimens were 152 mm in width and 254 mm in height and had 2#5 longitudinal bars at bottom, 2Ø8 bars at top, Ø8@80 mm stirrups in shear span and Ø8@333.33 mm stirrups in the constant moment region. Accelerated corrosion was imposed to longitudinal bars of all specimens except the sound one. Shear span ratio of the beams is 2.54. Four-point bending loading was applied to the system.

Figure 3-9 Geometry of Maaddawy’s test beam
The finite element model based on Maaddawy’s specimen was developed. All the properties including material and geometry properties applied to the model (Figure 3-9). As the corrosion level of steel bars for both longitudinal and transverse reinforcements are higher than 10%, the reinforcing bars modeled fully un-bonded to concrete in corroded beam specimens, CN-310.

<table>
<thead>
<tr>
<th>Column Geometry</th>
<th>B (mm)</th>
<th>H (mm)</th>
<th>L (mm)</th>
<th>Cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>152</td>
<td>254</td>
<td>3200</td>
<td>25</td>
</tr>
<tr>
<td>Concrete</td>
<td>41</td>
<td>0.00243</td>
<td>0.0038</td>
<td>3.99</td>
</tr>
<tr>
<td>Longitudinal Steel Bars</td>
<td>2#5</td>
<td>1.04%</td>
<td>450</td>
<td>0.00225</td>
</tr>
<tr>
<td>Transverse Steel Bars</td>
<td>Ø8@80mm, 333.33mm</td>
<td>1.94%, 0.47%</td>
<td>340</td>
<td>0.0017</td>
</tr>
<tr>
<td>Shear Span to Depth Ratio</td>
<td>2.54</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Code</th>
<th>Virgin</th>
<th>CN-310</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrosion Loss Ratio (%)</td>
<td>Longitudinal</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 3-10 shows the lateral load- lateral displacement and crack pattern of Maaddawy’s un-corroded test beam. Yielding of tension reinforcement had occurred prior to concrete cover crushing started. The whole beam is fully cracked due to small shear span to depth ratio and finally the beam analysis stopped.
Figure 3-10 Lateral Load-displacement plot and Crack pattern of Maaddawy’s test beam (Virgin)

Figure 3-11 and Table 3-6 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is fairly close to experimental data and they are in reasonable agreement. But the ultimate displacement of FEA is far away of experimental one. Both beams experienced flexural failure.
Table 3-6 Comparison between experimental and FEA results; Maaddawy’s test beam (Virgin)

<table>
<thead>
<tr>
<th></th>
<th>$V_y (kN)$</th>
<th>$d_y (mm)$</th>
<th>$V_u (kN)$</th>
<th>$d_u (mm)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>67.2</td>
<td>15.63</td>
<td>75</td>
<td>73.33</td>
</tr>
<tr>
<td>FEM Results</td>
<td>63.60</td>
<td>14.24</td>
<td>71.23</td>
<td>24.81</td>
</tr>
</tbody>
</table>

Figure 3-11 Lateral force- lateral displacement curves of Maaddawy’s test sound (virgin); experimental vs. FEM model

Figure 3-12 shows the lateral load- lateral displacement and crack pattern of Maaddawy’s corroded test beam, CN-310. Corroded tension reinforcement yielded prior to concrete cover crushing started. Similar to un-corroded column, small shear span to depth ratio caused the cracks develop along the full length of corroded beam. Finally, the beam analysis stopped because of large number of flexural cracks, which result in un-convergence of model.

Figure 3-13 and Table 3-7 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is fairly close to experimental data and have reasonable agreement. But the ultimate displacement of FEA is far away of experimental one. Both beams experienced flexural failure.
Figure 3-12 Lateral Load-displacement plot and Crack pattern of Maaddawy’s test beam CN-310

Figure 3-13 Lateral force- lateral displacement curves of Maaddawy’s test beam CN-310; experimental vs. FEM model
Table 3-7 Comparison between experimental and FEA results; Maaddawy’s test beam CN-310

<table>
<thead>
<tr>
<th></th>
<th>( V_y ) (kN)</th>
<th>( d_y ) (mm)</th>
<th>( V_u ) (kN)</th>
<th>( d_u ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>44.16</td>
<td>11.74</td>
<td>53.27</td>
<td>58.96</td>
</tr>
<tr>
<td>FEM Results</td>
<td>48.49</td>
<td>10.69</td>
<td>50.86</td>
<td>26.73</td>
</tr>
</tbody>
</table>

3.5.1.2 Gong (2009)

Gong (2009) conducted combined lateral cyclic and constant axial loading test on fourteen RC columns to investigate the effect of combined CFRP and steel jacket retrofitting system on corroded RC columns. The specimens had cross-section of 200 mm x 200 mm and clear height of 1500 mm; and reinforced with 4\( \Phi 14 \) mm longitudinal bars and \( \Phi 8 \) mm@100 mm lateral hoops. Applying lateral cyclic load at mid-span of un-corroded columns, they found that by increasing the lateral load, flexural cracks were developing. After yielding of longitudinal bars, diagonal shear cracks appeared and finally the column failed in shear. Same loading on corroded columns showed that by increasing the lateral load, longitudinal cracks due to corrosion developed and followed by flexural cracks. Finally, complete spalling of concrete cover due to de-bonding between concrete cover and core caused the failure of corroded columns. Shear span ratio for this column is 2.5.

The finite element model based on Gong’s specimen was developed. All the properties including material and geometry properties applied to the model (Figure 3-14). Cyclic lateral loading is replaced by monotonic lateral loading. As the corrosion level of steel bars are higher than 10%, the reinforcing bars were modeled fully un-bonded to concrete in corroded column specimens.
### Table 3-8 Geometry and material properties of Gong’s test columns

<table>
<thead>
<tr>
<th>Column Geometry</th>
<th>$B$ (mm)</th>
<th>$H$ (mm)</th>
<th>$L$ (mm)</th>
<th>Cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>200</td>
<td>200</td>
<td>500</td>
<td>30</td>
</tr>
<tr>
<td>Un-confined Concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f'_c$ (MPa)</td>
<td>44.8</td>
<td>0.00255</td>
<td>0.0038</td>
<td>4.17</td>
</tr>
<tr>
<td>$\varepsilon_{c0}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{u0}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_r$ (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confined Concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f'_{cc}$ (MPa)</td>
<td>49.06</td>
<td>0.00376</td>
<td>0.0068</td>
<td>4.36</td>
</tr>
<tr>
<td>$\varepsilon_{c1}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{u1}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$f_r$ (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal Steel Bars</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>1.54%</td>
<td>384.77</td>
<td>0.00192</td>
<td>0.1282</td>
</tr>
<tr>
<td>$f_y$ (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{sy}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{su}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse Steel Bars</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\rho_v$</td>
<td>2.87%</td>
<td>326.95</td>
<td>0.00163</td>
<td>0.1447</td>
</tr>
<tr>
<td>$f_y$ (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{sy}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\varepsilon_{su}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Span to Depth Ratio</td>
<td></td>
<td></td>
<td></td>
<td>2.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Code</th>
<th>A0</th>
<th>B3</th>
<th>C2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrosion Loss Ratio (%)</td>
<td>0</td>
<td>16.8</td>
<td>11.49</td>
</tr>
<tr>
<td>Axial Load Ratio ($P/A_g \cdot f'_c$)</td>
<td>23.4%</td>
<td>23.4%</td>
<td>16.7%</td>
</tr>
</tbody>
</table>
Figure 3-15 Lateral Load-displacement plot and Crack pattern of Gong’s test column A0

Figure 3-15 shows the lateral load- lateral displacement and crack pattern of Gong’s un-corroded test column, A0. Yielding of tension reinforcement started prior to concrete cover crushing. The column experiences flexural and shear cracks to large amount of axial load and small shear span to depth ration. Finally, the column analysis stopped because of large number of flexural and shear cracks, which result in un-convergence of model.

Figure 3-16 and Table 3-9 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is close to experimental data and are in reasonable agreement. Both columns experienced flexural-shear failure.

Table 3-9 Comparison between experimental and FEA results; Gong’s test column A0

<table>
<thead>
<tr>
<th></th>
<th>$V_{le}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental Results</td>
<td>190.87</td>
</tr>
<tr>
<td>FEM Results</td>
<td>192.95</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Status</th>
<th>Displacement (mm)</th>
<th>Lateral Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yielding of longitudinal tension bars</td>
<td>3.300</td>
<td>187.87</td>
</tr>
<tr>
<td>Beginning of concrete crushing</td>
<td>4.590</td>
<td>190.97</td>
</tr>
<tr>
<td>Maximum load and displacement</td>
<td>5.330</td>
<td>192.95</td>
</tr>
</tbody>
</table>
Figure 3-16 Lateral force-lateral displacement curves of Gong’s test columns A0; experimental vs. FEM model

Figure 3-17 shows the lateral load-lateral displacement and crack pattern of Gong’s corroded test column, B3. Yielding of tension reinforcing bars started prior to concrete cover crushing.

Figure 3-17 Lateral Load-displacement plot of Gong’s test column B3
Figure 3-18 and Table 3-10 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is fairly close to experimental data and are in reasonable agreement. Both columns experienced flexural-shear failure.

![Figure 3-18 Lateral force- lateral displacement curves of Gong’s test columns B3; experimental vs. FEM model](image)

<table>
<thead>
<tr>
<th></th>
<th>Experimental Results</th>
<th>FEM Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{ud} ,(kN)$</td>
<td>173.20</td>
<td>179.34</td>
</tr>
</tbody>
</table>

Figure 3-19 shows the lateral load- lateral displacement and crack pattern of Gong’s corroded test column, C2. Yielding of corroded tension reinforcing bars started prior to concrete cover crushing. Axial load ratio in this column is less than the other columns, which resulted in significant shear cracks along the column height, followed by flexural cracks. The column analysis stopped because of large number of cracks, which result in un-convergence of model.
Figure 3-20 and Table 3-11 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is fairly close to experimental data and have reasonable agreement. Both columns experienced flexural-shear failure.

Table 3-11 Comparison between experimental and FEA results; Gong’s test column C2

<table>
<thead>
<tr>
<th></th>
<th>$V_{ul} (kN)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental Results</td>
<td>167.80</td>
</tr>
<tr>
<td>FEM Results</td>
<td>153.86</td>
</tr>
</tbody>
</table>
According to provided pictures, the concrete cover is not completely spalled off (Figure 3-21). In all corroded specimens, the corrosion level is less than 20% and therefore, concrete cover has contribution on load carrying capacity of corroded columns.
3.5.1.3 Ou et al. (2012)

Ou et al. (2012) carried out cyclic loading on five large-scale beams to investigate cyclic performance of corroded beams. The specimens with 30 cm in width and 50 cm in height had 5#5 longitudinal bars at top and bottom and #3@10 cm stirrups. The cantilever beams designed according to ACI 318 and transverse reinforcement designed in a way that flexural failure mode was controlling. Accelerated corrosion was imposed to all specimens except the sound one. Cyclic loading was applied to the free end of the cantilever and the results showed that because of rupture of transverse reinforcement due to corrosion, longitudinal bars buckled and failure mode switched from flexural failure mode to flexural-shear one. Shear span ratio of the beams is 3.

A finite element model based on Ou’s specimen was developed. All the properties including material and geometry properties applied to the model (Figure 3-22). Cyclic lateral loading is replaced by monotonic lateral loading. As the corrosion level of steel bars for both longitudinal and transverse reinforcements are higher than 10%, the reinforcing bars modeled fully un-bonded to concrete in corroded beam specimens, B-150.
Table 3-12 Geometry and material properties of Ou’s test beams

<table>
<thead>
<tr>
<th>Column Geometry</th>
<th>$B$ (mm)</th>
<th>$H$ (mm)</th>
<th>$L$ (mm)</th>
<th>Cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$f'_{c}$ (MPa)</td>
<td>$\varepsilon_{c0}$</td>
<td>$\varepsilon_{u0}$</td>
<td>$f_{c}$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>0.00255</td>
<td>0.0038</td>
<td>4.17</td>
</tr>
<tr>
<td>Longitudinal Steel Bars</td>
<td>$\rho_{x}$</td>
<td>$f_{y}$ (MPa)</td>
<td>$\varepsilon_{xy}$</td>
<td>$\varepsilon_{su}$</td>
</tr>
<tr>
<td></td>
<td>1.33%</td>
<td>488</td>
<td>0.00224</td>
<td>0.1102</td>
</tr>
<tr>
<td>Transverse Steel Bars</td>
<td>$\rho_{v}$</td>
<td>$f_{y}$ (MPa)</td>
<td>$\varepsilon_{xy}$</td>
<td>$\varepsilon_{su}$</td>
</tr>
<tr>
<td></td>
<td>1.07%</td>
<td>488</td>
<td>0.00224</td>
<td>0.1102</td>
</tr>
<tr>
<td>Shear Span to Depth Ratio</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Code</th>
<th>B-0</th>
<th>B-150</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrosion Loss Ratio (%)</td>
<td>Longitudinal</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Transverse</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 3-23 shows the lateral load- lateral displacement and crack pattern of Ou’s uncorroded test beam, B0. Yielding of tension reinforcement had occurred prior to concrete cover crushing started; followed by yielding of compression reinforcing bars. The whole beam is fully cracked due to small shear span to depth ratio and finally the beam analysis stopped.
Figure 3-23 Lateral Load-displacement plot and Crack pattern of Ou’s test beam B-0

Figure 3-24 and Table 3-13 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is fairly close to experimental data and are in reasonable agreement. But the ultimate displacement of FEA is far away of experimental one. Both beams experienced flexural failure.

Table 3-13 Comparison between experimental and FEA results; Ou’s test beam B-0

<table>
<thead>
<tr>
<th></th>
<th>$V_y \text{ (kN)}$</th>
<th>drift y (%)</th>
<th>$V_u \text{ (kN)}$</th>
<th>drift u (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>109</td>
<td>0.61</td>
<td>155</td>
<td>4.4</td>
</tr>
<tr>
<td>FEM Results</td>
<td>110.10</td>
<td>0.42</td>
<td>149.07</td>
<td>2.42</td>
</tr>
</tbody>
</table>

Figure 3-24 Lateral force- lateral displacement curves of Ou’s test beam B-0; experimental vs. FEM model

Figure 3-25 shows the lateral load- lateral displacement and crack pattern of Ou’s corroded test beam, B-150. Tension reinforcement yielded prior to concrete cover crushing
started. Similar to un-corroded column, small shear span to depth ratio caused the cracks to develop along the full length of corroded beam. Finally the beam analysis stopped because of large number of flexural and shear cracks, which result in un-convergence of model.

Figure 3-25 Lateral Load-displacement plot of Ou’s test beam B-150

Figure 3-26 and Table 3-14 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is fairly close to experimental data and are in reasonable agreement. But the ultimate displacement of FEA is far away of experimental one. Both beams experienced flexural-shear failure.

<table>
<thead>
<tr>
<th>Status</th>
<th>Drift Ratio (%)</th>
<th>Lateral Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yielding of Longitudinal Tension Bars</td>
<td>0.365</td>
<td>92.87</td>
</tr>
<tr>
<td>Beginning of Concrete Crushing</td>
<td>1.123</td>
<td>117.32</td>
</tr>
<tr>
<td>Maximum Load</td>
<td>1.905</td>
<td>117.87</td>
</tr>
</tbody>
</table>

Table 3-14 Comparison between experimental and FEA results; Ou’s test beam B-150

<table>
<thead>
<tr>
<th></th>
<th>$V_y$ (kN)</th>
<th>$\text{drift}_y$ (%)</th>
<th>$V_u$ (kN)</th>
<th>$\text{drift}_u$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental Results</td>
<td>92</td>
<td>0.5</td>
<td>120</td>
<td>2.6</td>
</tr>
<tr>
<td>FEM Results</td>
<td>92.87</td>
<td>0.37</td>
<td>117.87</td>
<td>1.90</td>
</tr>
</tbody>
</table>
According to provided pictures, the concrete cover is not completely spalled off (Figure 3-27). In corroded specimen, the corrosion level is less than 20% and therefore, concrete cover has contribution on load carrying capacity of corroded beam.
3.5.1.4 Meda et al. (2014)

Meda et al. (2014) conducted combined lateral cyclic and constant axial loading test on two RC columns to investigate the effect of corrosion on corroded RC columns. The specimens had cross-section of 300 mm x 300 mm and clear height of 1500 mm; and reinforced with 4\(\Phi_{16}\) mm longitudinal bars and #8 mm@300 mm lateral hoops. Applying lateral cyclic load at the end of un-corroded columns, they found that by increasing the lateral load, flexural cracks were developing. After complete yielding of longitudinal bars and large deformation of compression bars due to buckling, the column experienced the maximum lateral load and then due to concrete crushing and cover spalling the test had been stopped. Same loading on corroded columns showed that by increasing the lateral load, longitudinal cracks due to corrosion developed and followed by flexural cracks. Finally, complete spalling of concrete cover due to de-bonding between concrete cover and core, crushing of concrete and buckling of corroded bars caused the failure of corroded columns. Shear span ratio for this column is 5.

A finite element model based on Meda’s specimen was developed. All the properties including material and geometry properties applied to the model (Figure 3-28). Cyclic lateral loading is replaced by monotonic lateral loading. As the corrosion level of steel bars are higher than 10%, the reinforcing bars modeled fully un-bonded to concrete in corroded column specimens.
Figure 3-28 Geometry of Meda’s test column

Table 3-15 Geometry and material properties of Meda’s test columns

<table>
<thead>
<tr>
<th>Column Geometry</th>
<th>B (mm)</th>
<th>H (mm)</th>
<th>L (mm)</th>
<th>Cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>300</td>
<td>300</td>
<td>1500</td>
<td>30</td>
</tr>
<tr>
<td>Un-confined Concrete</td>
<td>$f'_c$ (MPa)</td>
<td>$\varepsilon_{\theta}$</td>
<td>$\varepsilon_{u0}$</td>
<td>$f_r$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.00170</td>
<td>0.0038</td>
<td>2.79</td>
</tr>
<tr>
<td>Confined Concrete</td>
<td>$f'_{cc}$ (MPa)</td>
<td>$\varepsilon_{\theta1}$</td>
<td>$\varepsilon_{u1}$</td>
<td>$f_r$ (MPa)</td>
</tr>
<tr>
<td></td>
<td>21.47</td>
<td>0.00232</td>
<td>0.0043</td>
<td>2.89</td>
</tr>
<tr>
<td>Longitudinal Steel Bars</td>
<td>$\rho_z$</td>
<td>$f_y$ (MPa)</td>
<td>$\varepsilon_{xy}$</td>
<td>$\varepsilon_{su}$</td>
</tr>
<tr>
<td>4016</td>
<td>0.89%</td>
<td>520</td>
<td>0.00260</td>
<td>0.0897</td>
</tr>
<tr>
<td>Transverse Steel Bars</td>
<td>$\rho_v$</td>
<td>$f_y$ (MPa)</td>
<td>$\varepsilon_{xy}$</td>
<td>$\varepsilon_{su}$</td>
</tr>
<tr>
<td>#8@300mm</td>
<td>0.34%</td>
<td>520</td>
<td>0.00260</td>
<td>0.0897</td>
</tr>
<tr>
<td>Shear Span to Depth Ratio</td>
<td></td>
<td></td>
<td></td>
<td>5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Code</th>
<th>UC</th>
<th>CC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrosion Loss Ratio (%)</td>
<td>0</td>
<td>20</td>
</tr>
<tr>
<td>Axial Load Ratio ($P/A_g f'_c$)</td>
<td>22.0%</td>
<td>22.0%</td>
</tr>
</tbody>
</table>
Figure 3-29 shows the lateral load-lateral displacement and crack pattern of Meda’s uncorroded test column, UC. Yielding of tension reinforcement started prior to concrete cover crushing. The column experiences flexural cracks to large amount of axial load and large shear span to depth ratio.

Figure 3-30 and Table 3-16 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load of FEA is close to experimental data and are in reasonable agreement. Both columns experienced flexural-shear failure.

| Table 3-16 Comparison between experimental and FEA results; Meda’s test column UC |
|-----------------------------------|---------|------------------|
|                                   | $V_u$ (kN) | $drift_u$ (%)    |
| Experimental Results             | 63       | 5                |
| FEM Results                      | 60.83    | 4.6              |
Figure 3-30 Lateral force-lateral displacement curves of Meda’s test columns UC; experimental vs. FEM model

Figure 3-31 shows the lateral load-lateral displacement and crack pattern of Meda’s corroded test column, CC. Yielding of tension reinforcing bars started at the same time of concrete cover crushing.

<table>
<thead>
<tr>
<th>Status</th>
<th>Drift Ratio (%)</th>
<th>Lateral Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Yielding of Longitudinal Tension Bars</td>
<td>1.342</td>
<td>48.12</td>
</tr>
<tr>
<td>2 Beginning of Concrete Cracking</td>
<td>1.342</td>
<td>48.12</td>
</tr>
<tr>
<td>3 Maximum Load</td>
<td>1.150</td>
<td>48.80</td>
</tr>
</tbody>
</table>

Figure 3-31 Lateral Load-displacement plot of Meda’s test column CC
Figure 3-32 and Table 3-17 show comparison between experimental and FE modeling lateral load-displacement curve and results. The maximum lateral load and drift ratio of FEA is fairly close to experimental data and are in reasonable agreement. Both columns experienced flexural failure.

![Figure 3-32 Lateral force- lateral displacement curves of Meda’s test column CC; experimental vs. FEM model](image)

| Table 3-17 Comparison between experimental and FEA results; Meda’s test column CC |
|-----------------------------------|-------|------------------|
|                                   | $V_u$ (kN) | $drift_u$ (%)    |
| Experimental Results              | 46.00   | 2.5              |
| FEM Results                       | 48.80   | 2.54             |

According to provided pictures, the concrete cover is not completely spalled off (Figure 3-33). In corroded specimen, the corrosion level is less than 20% and therefore, concrete cover has contribution on load carrying capacity of corroded column.
Vu et al. (2016) also performed finite element analysis on DIANA to verify Meda’s experimental data. They used a lower yielding stress for corroded bars which resulted in the following lateral load-displacement curve. Figure 3-16 shows comparison between experimental and FE modeling lateral load-displacement curve in both DIANA and current study (ABAQUS), using the same yielding stress. The maximum lateral load of both FEA is fairly close and are in reasonable agreement.
3.6 Summary and Conclusion

Finite element model of RC columns subjected to axial and lateral loading is in good agreement with experimental data to estimate the maximum lateral load capacity of columns. However, FEM cannot predict ultimate lateral displacement of columns. Although FE model does not provide any rough estimation of ductility, it can show the load carrying capacity of RC columns very well (Figure 3-35).

![Figure 3-35 Comparison of lateral capacity of corroded elements, Experimental vs. FEA results](image-url)
4 LATERAL STRENGTH EVALUATION OF CORRODED COLUMNS

4.1 Introduction

This chapter investigates response of severely corroded columns subjected to transverse loading in addition to compressive axial load, based on finite element analysis. Columns are classified into three main groups; (1) columns with corrosion in compression side of the cross-section, (2) columns with corrosion in tension side of the cross-section, and (3) columns with corrosion on all sides of the cross-section.

Columns in each group are also sorted based on their shear span to depth ratio. Shear span to depth ratio is the main factor to define mode of failure in columns. Generally, the mode of failure for L/d < 2 is shear; while for L/d > 4 flexural mode of failure dominates. For 2 < L/d < 4, both shear and flexural strength demands are equal and the mode of failure is uncertain, reflected to flexural-shear mode of failure (Wan et al., 2010). However, experimental data show that only 63% of columns with L/d < 2 failed in shear; and only 51% of columns with L/d > 4 failed in flexure. Therefore, there are no certain boundaries for L/d to specify failure mode and obviously, shear span to depth ratio cannot be enough to define failure mode of columns (Qi et al., 2013).

In this chapter, lateral behavior of corroded columns is studied of each group based on different parameters that affect column failures.

4.2 Objectives and Scopes

In this study, reinforced concrete columns are modeled as a cantilever. Axial and lateral loads are applied to the free end of the column as shown in Figure 4-1. All sections are 24 in x
24 in with the same transverse and one of the three amounts of longitudinal reinforcement. It is important to mention that corrosion level in this study indicates steel bar area loss.

![Figure 4-1 (a) Side View of the Column; (b) Cross-section of the Column](image)

The following parameters are the primary variables in this study:

- Location of corrosion within the cross-section (Compression-side corroded, Tension-side corroded, All-sides corroded)
- Corrosion level (CR=25%, 30%, 35%, 40%, 45%, 50%)
- Length of corroded section along the column height (1H=24in, 2H=48in)
- Axial load ratio \( NR = \frac{P}{f'cA_g} = 0\%, 5\%, 15\%, 25\% \)

- Compressive strength of concrete \( f'_c = 4\, ksi, 7\, ksi \)

- Steel reinforcing ratio \( \rho = 2\%, 3\%, 4\% \)

- Shear span to depth ratio \( L/d=2.5, 5 \).

As discussed in section 2.7, when corrosion level in the main steel bars is 25\%, stirrups would have fractured and concrete cover spalls off completely. Therefore, concrete cover and stirrups are removed at corroded locations. For this level of corrosion, there is no bond between corroded bars and concrete. So, corroded compression bars have potential to buckle at corroded height zone.
Figure 4-2 Cases Studied in Finite Element Analysis
4.3 Finite Element Analysis of Corroded Columns

The group of columns with L/d=5 have a length of 10 feet. Flexural failure mode is expected for un-corroded columns due to high shear span to depth ratio. The group of columns with L/d=2.5 have a length of 5 feet. Flexural-shear failure mode is expected for un-corroded columns due to lower shear span to depth ratio.

Transverse loading of the columns is displacement control loading with maximum increments of 0.01 inch. If loading has not been stopped by un-convergence problems, it is continued to 5% drift ratio or the displacement at which 85% of maximum lateral capacity is reached.

Three different corrosion locations in column cross-section are studied in this chapter; compression-side corroded, tension-side corroded and all-sides corroded column.

4.3.1 Compression-side Corroded Columns

When compression side of the section is corroded, compression bars tend to buckle. Then, the force carried by corroded compression bars is limited to buckling force. As there is no stirrup and concrete cover at compression zone, concrete core is considered unconfined. Less compressive force in compression bars with less concrete compression zone area and unconfined concrete core lead the column to carry less lateral load in comparison with un-corroded column. Failure mode for all corroded columns with L/d=5 was flexural-failure similar to un-corroded specimens. Failure mode for compression-side corroded columns with L/d=2.5 depends on axial load.

Figure 4-3 shows lateral capacity-displacement diagram of compression-side corroded column with L/d=5 compared to un-corroded column. Applying lateral load, corroded
compressive bars start to buckle, as there is no bond between the corroded bars and concrete core. When the force carried by compressive bars is limited to buckling force, compressive concrete should carry the remaining compression loads. On the other hand, higher lateral displacement causes tension bars start to yield. Yielding of tensile bars increases the strain at tensile bars suddenly. Due to strain compatibility and constant forces at the section, compression zone gets smaller; which means neutral axis shifts toward compression zone. The higher lateral load, compression zone area for concrete becomes smaller which leads the concrete to carry more loads and finally crushing of compressive concrete happens. The more axial load, buckling of corroded bars happens earlier. In some cases, buckling occurs even under axial load.

In general, the displacement at which the column reaches its maximum lateral capacity does not change significantly due to corrosion. In overall, yielding of tension bars in compression-side corroded columns occur later than bars in un-corroded models. But crushing of concrete in compression zone happens at lower lateral displacement in compression-side corroded columns than un-corroded ones. Therefore, in compression-side corroded columns, the distance between yielding of tension bars and crushing of compressive concrete is lower, which makes the compression-side corroded columns to have less ductility. Reduction in ductility of corroded columns is seen more in columns with low concrete compressive strength ($f'_c$) and higher axial loads (Appendix 1).

As mentioned above, crushing of concrete in compression zone happens at lower lateral displacement in compression-side corroded columns than un-corroded ones. In some cases, under high axial load, crushing of concrete even happens before yielding of tension bars; which put the column status in compression-control zone at P-M interaction diagram. So the columns in
this zone, have a relative brittle failure mode. This kind of failure is seen in columns with higher reinforcing ratio subjected to high axial load (Appendix 1).

Figure 4-3 Lateral capacity- displacement diagram of compression-side corroded column compared to un-corroded column (L/d=5)

Figure 4-4 shows the crack pattern of un-corroded and compression-side corroded columns with L/d=5 at ultimate limit state. Applying lateral load, flexural cracks start to occur at tension side of the column at plastic hinge zone. The higher lateral load, cracks progress in depth and develop more in the height of the column and make the compression zone smaller and smaller and finally crushing of compressive concrete happens. Crack patterns for un-corroded columns and corroded ones are very similar and all show flexural failure.
The initial stiffness of compression-side corroded columns with L/d=5 decreases in comparison with un-corroded columns. This is majorly because of loss of concrete cover at compression zone. Area of compression zone in column cross-section has an important role in carrying axial load as well as compressive load due to bending moment that lateral displacement produces. Corrosion level does not have significant effect on initial stiffness of corroded columns.

Figure 4-5 shows lateral capacity-displacement diagram of compression-side corroded column with L/d=2.5 compared to un-corroded column. Applying lateral load, buckling of corroded compression bars occurs first. Based on axial load ratio, yielding of tension bars occurs before or after crushing of compressive concrete. Yielding of transverse reinforcement happens
before the peak of the curve for all corroded columns with \(L/d=2.5\). The more axial load, buckling of corroded bars happens earlier. In some cases, buckling occurs even under axial load.

![Lateral capacity-displacement diagram of compression-side corroded column compared to un-corroded column (L/d=2.5)](image)

Mode of failure of un-corroded columns is flexural-shear. Behavior of corroded columns with high axial load is similar to un-corroded columns. However, under low axial load, mode of failure of corroded columns may change. Under low axial loads and high corrosion level (especially when axial load is zero and corrosion level is 50%), corroded column shows more flexural behavior. This is because of less contribution of compression zone including concrete.
and buckled corroded bars, which shifts the neutral axis toward compression zone and makes the tension bars yield. Therefore, under very low axial load, compression-side corroded columns have more ductility compared with un-corroded columns (Appendix 1).

Figure 4-6 shows the crack pattern of un-corroded and compression-side corroded columns with L/d=2.5 at ultimate limit state. Applying lateral load, cracks start to occur. The higher lateral load, shear cracks as well as flexural cracks progress in depth and develop more in the height of the column and make the compression zone smaller and smaller and finally crushing of compressive concrete happens.

![Figure 4-6 Crack pattern of un-corroded and compression-side corroded columns (L/d=2.5); (a to c) respectively axial load ratios of 5%, 15% and 25%.](image)
Crushing of concrete in compression zone happens at lower lateral displacement in compression-side corroded columns than un-corroded ones. In some cases, under high axial load, crushing of concrete even happens before yielding of tension bars. The compression-side corroded columns subjected to relatively high axial load have a similar failure mode to that of un-corroded columns. In general, the displacement at which the column reaches its maximum lateral capacity does not change significantly due to corrosion (Appendix 1).

The initial stiffness of compression-side corroded columns with $L/d=2.5$ decreases in comparison with un-corroded columns. However, the corrosion level does not have significant effect on initial stiffness of corroded columns.

4.3.2 Tension-side Corroded Columns

There is no bond between tensile reinforcing bars and concrete in tension-side corroded columns. Although tensile strain is lower for un-bonded bars than bonded reinforcing bars, tensile bars in tension-side corroded columns could yield and even enter strain hardening zone. This is because of smaller cross-sectional area and lower yielding stress of corroded bars. Therefore, the tensile force carried by corroded tension bars is reduced in general. Concrete core is considered confined in tension-side corroded columns. Removing concrete cover in tension side does not affect the moment capacity of the column, but reduces the compressive capacity. Therefore, under constant axial load, neutral axis moves toward compression zone in the absence of tensile concrete cover. This causes the corroded tensile bars yield prematurely, which results in carrying less lateral load. Failure mode for all corroded columns with $L/d=5$ was flexural-failure as un-corroded specimens. Failure mode for tension-side corroded columns with $L/d=2.5$ depends on steel reinforcing ratio.
Figure 4-7 shows the lateral capacity- displacement diagram of tension-side corroded column with L/d=5 compared to un-corroded column. Applying lateral load, tension bars start to yield and finally crushing of compressive concrete happens. Initial lateral displacement in the diagrams is because of un-symmetrical cross section of column under axial load. In some cases, buckling of corroded tension bars occurs under axial load. As the buckling is elastic, applying higher lateral load makes the buckled bars become straight and then corroded bars start to participate in load carrying capacity of column.

![Figure 4-7 Lateral capacity- displacement diagram of tension-side corroded column compared to un-corroded column (L/d=5)](image-url)
In overall, de-bonding of tensile corroded bars does not prevent premature yielding of corroded bars; however, de-bonding may delay yielding of corroded bars to some extent, when the length of corroded zone is higher. Loss of concrete cover in tension zone makes the neutral axis shift toward compression zone. On the other hand, corroded tensile bars could carry less tensile force than tensile bars in un-corroded columns. Therefore, shifting of neutral axis toward compression zone happens gradually which means tension-side corroded columns could maintain their capacity more. Crushing of concrete in compression zone occurs at larger lateral displacement in tensile-corroded columns than un-corroded ones; except in columns with high axial load, crushing happens at almost same lateral displacement for both corroded and un-corroded columns. Considering all above-mentioned observations, ductility of columns which are corroded in tension side is more in general (Appendix 2).

When the tension-side corroded column is subjected to lower axial load, the displacement at which the column reaches its maximum lateral capacity is greater than the corresponding displacement in an un-corroded column. However, in high axial loads, the displacement at which the column reaches its maximum lateral capacity does not change significantly due to corrosion. Corrosion on tension side of the column does not affect the initial stiffness of column with L/d=5, as the absence of concrete cover at tension zone does not affect the lateral stiffness of columns.

Figure 4-8 shows the crack pattern of un-corroded and tension-side corroded columns with L/d=5 at ultimate limit state. Applying lateral load, flexural cracks start to occur at tension side of the column at plastic hinge zone. The higher lateral load, cracks progress in depth and develop more in the height of the column and make the compression zone smaller and smaller.
and finally crushing of compressive concrete happens. Crack patterns for un-corroded columns and corroded ones are very similar and all show flexural failure.

Figure 4-8 Crack pattern of un-corroded and tension-side corroded columns (L/d=5); (a to c) respectively axial load ratios of 5%, 15% and 25%.

Figure 4-9 shows the lateral capacity- displacement diagram of tension-side corroded column with L/d=2.5 compared to un-corroded column. The first four graphs and the last four graphs represent tensile-corroded columns, respectively with steel reinforcing ratio equal to 3% and 4%. Applying lateral load, tension bars start to yield, then crushing of compressive concrete happens followed by yielding transverse reinforcement. Initial lateral displacement in the diagrams is because of un-symmetrical cross section of column under axial load. In some cases, buckling of corroded tension bars occurs under axial load. As the buckling is linear, applying
higher lateral load makes the buckled bars become straight and then corroded bars start to participate in load carrying capacity of column.

Premature yielding of corroded bars, and late crushing of compressive concrete in tension-side corroded columns with L/d=2.5 is similar to corroded columns with L/d=5. However, mode of failure is different and depends on steel reinforcing ratio. For columns with reinforcing ratio equal to 3%, mode of failure is generally flexural mode. This is better seen at columns with lower axial loads and it is because of premature yielding of corroded tension bars, which delays crushing of concrete. So the columns could develop their flexural capacity before reaching the shear capacity. Tension-side corroded columns with reinforcing ratio equal to 4%, have different behavior. Premature yielding of corroded bars and late crushing of compressive concrete are not enough for columns to develop their flexural capacity. High steel reinforcing ratio increases the flexural capacity while the shear capacity of columns is almost constant. Therefore, the columns fail in shear before reaching their flexural capacity. Therefore, mode of failure of tension-side corroded columns with reinforcing ratio equal to 4% is flexural-shear failure, in general (Appendix 2). Corrosion on tension side of the column does not affect the initial stiffness of column with L/d=2.5, as well. Ductility of tension-side corroded columns with L/d=2.5 is similar to corresponding columns with L/d=5 and it is more than un-corroded columns in general. Tension-side corroded columns with higher corrosion levels subjected to lower axial load, are more ductile.
Figure 4-9 Lateral capacity-displacement diagram of tension-side corroded column compared to un-corroded column 
(L/d=2.5)
Un-corroded Columns

Tension-side Corroded Columns

$\rho=3\%$

Tension-side Corroded Columns

$\rho=5\%$

Tension-side Corroded Columns

$\rho=4\%$

(a)                         (b)     (c)

Figure 4-10 Crack pattern of un-corroded and tension-side corroded columns (L/d=2.5); (a to c) respectively axial load ratios of 5%, 15% and 25%.

Figure 4-10 shows the crack pattern of un-corroded and tension-side corroded columns with L/d=2.5 at ultimate limit state. Upon application of the lateral load, cracks start to occur. The higher lateral load, shear cracks as well as flexural cracks progress in depth and develop more in the height of the column and make the compression zone smaller and smaller and finally crushing of compressive concrete happens. Mode of failure of un-corroded columns and tension-
side corroded columns with high steel reinforcing ratio is flexural-shear and mode of failure of tension-side corroded columns with low steel reinforcing ratio is flexural.

4.3.3 All-sides Corroded Columns

When corrosion occurs at all sides of the column, concrete cover at all sides of the section is removed. So there is a significant decrease in concrete cross-sectional area of column. Concrete cover has almost 25% area of the whole section. Therefore, compressive strength of column decreases drastically. There is no bond between corroded reinforcing bars and concrete. Although tensile strain is lower for un-bonded bars than bonded reinforcing bars, tensile bars could yield and even enter strain hardening zone because of smaller cross-sectional area and lower yielding stress of corroded bars. Therefore, the tensile force carried by corroded tension bars reduces. On the other side, compressive corroded bars are subjected to buckle. The ultimate force in compression bars is controlled by buckling force. Concrete core is considered unconfined because of existence of no external stirrups and no concrete cover. Less compressive strength due to cover loss, reduced ultimate forces tensile bars could carry due to steel area loss, and de-bonding and limited force carried by compression bars due to area loss and buckling result in significant decrease in load carrying capacity of all-sides corroded bars. Failure mode for all corroded columns with L/d=5 was flexural-failure as un-corroded specimens. Failure mode for all-sides corroded columns with L/d=2.5 depends on axial load.

Figure 4-11 shows the lateral capacity- displacement diagram of all-sides corroded column with L/d=5 compared to un-corroded column. Applying lateral load, the first layer of corroded bars in compression side starts to buckle. Based on axial load ratio, the second layer of compression bars could buckle as well. In lower axial load, tension bars start to yield and finally crushing of compressive concrete happens. When the axial load is higher, crushing may occur
before yielding of tension bars. In some cases, buckling of corroded bars occurs under axial load. As the buckling is elastic, applying higher lateral load makes the buckled bars in tension side become straight and then corroded bars start to participate in load carrying capacity of column.

![Figure 4-11 Lateral capacity-displacement diagram of all-sides corroded column compared to un-corroded column (L/d=5)](image)

Response of all-sides corroded columns regarding ductility is different in various axial load ratios (Appendix 3). The all-sides corroded columns behave similar to tension-side corroded columns when they are subjected to lower axial loads. There is a premature yielding of corroded tension bars and delay in crushing of compressive concrete and therefore, the ductility increases
relatively. When all-sides corroded columns are subjected to high axial load, they behave similar to compression-side corroded bar. Delay in yielding of tension bars and premature crushing of compressive concrete causes the corroded columns to have less ductility. In some cases, crushing of concrete even happens earlier than yielding of tension bars which puts the column in compressive-failure zone in P-M interaction diagram and column has a relatively brittle failure.

When all-sides corroded column is subjected to lower axial load, the displacement at which the column reaches its maximum lateral capacity is greater than the corresponding displacement in an un-corroded column. However, in high axial loads, the displacement at which the column reaches its maximum lateral capacity does not change significantly due to corrosion (Appendix 3). This behavior is similar to tension-side corroded bars which means the displacement at which maximum lateral capacity is reached, is controlled by corroded tensile bars.

The initial stiffness of all-sides corroded columns with L/d=5 decreases in comparison with un-corroded columns, primarily because of loss of concrete cover at all sides of the section and secondly, because of lower cross-sectional area of all corroded bars. The high corrosion level, initial stiffness of all-sides corroded columns decreases.

Figure 4-12 shows the crack pattern of un-corroded and all-sides corroded columns with L/d=5 at ultimate limit state. Applying lateral load, flexural cracks start to occur at tension side of the column at plastic hinge zone. The higher lateral load, cracks progress in depth and develop more in the height of the column and make the compression zone smaller and smaller and finally crushing of compressive concrete happens. Crack patterns for un-corroded columns and corroded ones are very similar and all show flexural failure.
Figure 4-12 Crack pattern of un-corroded and all-sides corroded columns \((L/d=5)\); (a to c) respectively axial load ratios of 5\%, 15\% and 25\%.

Figure 4-13 shows the lateral capacity- displacement diagram of all-sides corroded column with \(L/d=2.5\) compared to un-corroded column. The first four graphs and the last four graphs represent tensile-corroded columns, respectively with steel reinforcing ratio equal to 3\% and 4\%. Applying lateral load, the first layer of bars in compression side starts to buckle. Based on axial load ratio, the second layer of compression bars could buckle as well. In lower axial load, tension bars start to yield and finally crushing of compressive concrete happens. Yielding of transverse reinforcement happens before the peak of the curve for all corroded columns with \(L/d=2.5\). When the axial load is higher, crushing may occur before yielding of tension bars. In some cases, buckling of corroded bars occurs under axial load. As the buckling is linear,
applying higher lateral load makes the buckled bars in tension side become straight and then corroded bars start to participate in load carrying capacity of column.

Mode of failure for all-sides corroded columns with L/d=2.5 depends on axial load. For columns with low axial load, mode of failure is generally flexural mode. This is because of less contribution of compression zone including concrete and buckled corroded bars, which shifts the neutral axis toward compression zone and makes the tension bars yield. Premature yielding of corroded tension bars delays crushing of concrete. So the columns could develop their flexural capacity before reaching the shear capacity. For columns with high axial load, column shows more flexural-shear behavior (Appendix 3).

Response of all-sides corroded columns regarding ductility is different in various axial load ratios. The all-sides corroded columns behave similar to tension-side corroded columns when they are subjected to lower axial loads. There is a premature yielding of corroded tension bars and delay in crushing of compressive concrete and therefore, the ductility increases relatively. When all-sides corroded column is subjected to high axial load, the column behaves similar to compression-side corroded bar. Delay in yielding of tension bars and premature crushing of compressive concrete causes the corroded column to have less ductility. In some cases, crushing of concrete even happens earlier than yielding of tension bars. Columns with higher corrosion levels subjected to lower axial load, are more ductile.
Figure 4-13 Lateral capacity-displacement diagram of all-sides corroded column compared to un-corroded column (L/d=2.5)
It was noticed at tension-side corroded columns that the displacement at which column reaches its maximum lateral capacity is controlled by yielding of corroded tension bars. As the all-sides corroded columns behave similar to tension-side corroded columns while subjected to low axial loads, the displacement at which the column reaches its maximum lateral capacity is greater than the corresponding displacement in an un-corroded column. Similarly, all-sides corroded column under high axial load responds same as compression-side corroded column. Therefore, the displacement at which the column reaches its maximum lateral capacity does not change significantly due to corrosion (Appendix 3).

The initial stiffness of all-sides corroded columns with L/d=2.5 decreases in comparison with un-corroded columns, primarily because of loss of concrete cover at all sides of the section and secondly, because of lower cross-sectional area of all corroded bars. The high corrosion level, initial stiffness of all-sides corroded columns decreases.

Figure 4-14 shows the crack pattern of un-corroded and all-sides corroded columns with L/d=2.5 at ultimate limit state. Applying lateral load, cracks start to occur. The higher lateral load, shear cracks as well as flexural cracks progress in depth and develop more in the height of the column and make the compression zone smaller and smaller and finally crushing of compressive concrete happens. Mode of failure of un-corroded columns and all-sides corroded columns with high axial load is flexural-shear and mode of failure of all-sides corroded columns with low axial load is flexural.
4.4 Summary and Conclusion

Finite element analysis of severely corroded reinforced concrete columns has been studied in detail in this chapter. Three groups of columns have been investigated based on their corrosion location on the cross-section (compression-side, tension-side and all-sides corroded).
Corroded columns with L/d=5 have the same flexural failure as un-corroded columns. Although un-corroded columns with L/d=2.5 fail in flexural-shear, corroded columns could fail in flexural-shear or flexural mode based on their axial load ratio and steel reinforcing ratio. Mode of failure of all-sides corroded and compression-side corroded columns with L/d=2.5 depends on axial load; columns subjected to lower axial load fail in flexure and columns with high axial load fail in flexural-shear mode. Failure mode of tension-side corroded columns depends on steel reinforcing ratio. Lower steel reinforcing ratio results in flexural failure and higher steel reinforcing ratio leads the corroded column fail in flexural-shear mode.

In general, ductility due to corrosion increases in tension-side corroded columns and decreases in compression-side corroded columns, regardless of L/d ratio. In all-sides corroded columns ductility depends on axial load. Mostly, corroded columns with low axial load experiences more ductility and corroded columns with high axial load have less ductility in comparison with un-corroded columns, regardless of L/d ratio. Ductility of all-sides corroded and tension-side corroded columns with high corrosion level and under low axial load is significantly higher than corresponding columns with lower corrosion level.

Initial stiffness of tension-side corroded columns does not change because of corrosion; however, corrosion reduces the initial stiffness of compression-side corroded columns and all-sides corroded columns. The high the corrosion level, the less initial stiffness is seen in all-sides corroded columns. Corrosion rate has the minimal effect on initial stiffness of compression-side corroded columns.

In general, the displacement at which the column reaches its maximum lateral capacity does not change significantly due to corrosion, except tension-side corroded and all-sides
corroded columns with very low axial load, for which the corroded column reaches its maximum lateral capacity in a higher lateral displacement.
5 EFFECT OF DIFFERENT PARAMETERS ON LATERAL CAPACITY OF SEVERELY CORRODED RC COLUMNS

5.1 Introduction

Three different corrosion locations in column cross-section are studied in this research; compression-side corroded, tension-side corroded and all-sides corroded column. Effect of different parameters on lateral capacity of columns in each group is investigated in the current chapter.

5.2 Compression-side Corroded Columns

Generally, lateral capacity reduction due to severe corrosion in compression-side corroded columns is 20 to 45% for columns with $L/d=5$ and 15 to 40% for column with $L/d=2.5$. As discussed in section 4.3.1, compression-side corroded columns with $L/d=5$ have the same flexural failure as un-corroded columns. Although un-corroded columns with $L/d=2.5$ fail in flexural-shear, mode of failure of compression-side corroded columns with $L/d=2.5$ depends on axial load; columns subjected to lower axial load fail in flexure and columns with high axial load fail in flexural-shear mode. So in general, corrosion has the same effect on lateral capacity of compression-side corroded columns subjected to low axial load with both $L/d=2.5$&5. There are several factors that affect the lateral capacity of compression-side corroded columns.

5.2.1 Effect of Corrosion Level

As shown in Figure 5-1 and Figure 5-2, high corrosion level results in less lateral capacity in corroded columns. Effect of corrosion level after corrosion is started, is more when the length of corroded zone is low. In longer corroded zone and/or low steel reinforcing ratio,
lateral capacity does not change significantly due to different corrosion levels. As the corroded buckled bars carry almost the same amount of force, the capacities are very close to each other.

As far as columns enter the severe corrosion level, corrosion level has insignificant effect on lateral strength of compression-side corroded columns. When corrosion level increases from 25% to 50%, the columns with 1H and 2H corroded length zone have an average of 5% and 2% reduction in lateral strength, respectively.

Figure 5-1 Effect of corrosion level and length of corroded zone on lateral capacity of compression-side corroded columns (L/d=5)
5.2.2 Effect of Length of Corroded Zone

As shown in Figure 5-1 through Figure 5-4, longer corroded zone results in less lateral capacity. When the length of corroded zone is doubled in compression-side corroded columns, the lateral capacity is reduced by about 5% in columns with L/d=5 and about 9% in columns with L/d=2.5. Effect of higher corroded length zone on lateral strength of compression-side corroded columns with higher $f'_{c}$ is even less. In most cases, lateral capacity reduction due to different lengths of corroded zone is independent of axial load ratio. As the corroded buckled bars carry almost the same amount of force, the difference in column capacities is insignificant.

In general, columns with longer corroded zones exhibit less ductility. The initial stiffness of compression-side corroded columns drops suddenly because of doubled length of corroded zone.
Figure 5-3 Effect of length of corroded zone on lateral capacity of compression-side corroded columns (L/d=5)
5.2.3 Effect of Compressive Strength of Concrete

Figure 5-5 shows effect of $f'_c$ on lateral capacity reduction (ratio of lateral capacity of corroded column to un-corroded column) of compression-side corroded columns. Compression-side corroded columns with different $f'_c$ have slight difference on lateral capacity reduction, especially on lower axial load ratios.
Figure 5-5 Effect of $f'_c$ on lateral capacity reduction of compression-side corroded columns (L/d=5)

5.2.4 Effect of Steel Reinforcing Ratio ($\rho$)

Figure 5-6 shows effect of $\rho$ on lateral capacity reduction (ratio of lateral capacity of corroded column to un-corroded column) of compression-side corroded columns with L/d=5. Compression-side corroded columns with different $\rho$ have slight difference on lateral capacity reduction, especially for column with $f'_c = 7\, ksi$. 
Figure 5-6 Effect of $\rho$ on lateral capacity reduction of compression-side corroded columns (L/d=5)

Figure 5-7 shows the effect of $\rho$ on lateral capacity reduction (ratio of lateral capacity of corroded column to un-corroded column) of compression-side corroded columns with L/d=2.5. Lateral capacity reduction in compression-side corroded columns with lower $\rho$ is more than columns with higher reinforcing ratio. Columns with higher steel reinforcing ratio have higher flexural capacity and tend to fail in shear; therefore, corrosion has less effect on these columns.

Figure 5-7 Effect of $\rho$ on lateral capacity reduction of compression-side corroded columns (L/d=2.5)
5.2.5 Effect of Axial Load

Figure 5-8 shows the effect of axial load on lateral capacity reduction of compression-side corroded columns. At high levels of axial load, a higher lateral capacity reduction in compression-side corroded columns is seen. Un-corroded columns with high axial load carry more loads than columns with low axial load, because all the studied columns are designed to be in tension-zone of P-M diagram. When the compressive bars are corroded, the columns with high axial load are in more critical situation as the corroded bars buckle soon and they have slight contribution to load carrying capacity of columns. Therefore, their lateral capacity compared to un-corroded column reduces much.
The average lateral capacity reduction due to increase of axial load in compression-side corroded columns with L/d=2.5 (19%) is less than columns with L/d=5 (25%).

5.2.6 Effect of Shear Span to Depth Ratio (L/d)

Figure 5-9 shows the effect of shear span to depth ratio on lateral capacity reduction of compression-side corroded columns. Due to corrosion in compression-side, columns with L/d=2.5 have less lateral capacity reduction than columns with L/d=5. This is more noticeable when the length of corroded zone is lower.

![Figure 5-9 Effect of L/d on lateral capacity reduction of compression-side corroded columns](image)

5.3 Tension-side Corroded Columns

Generally, lateral capacity reduction due to severe corrosion in tension-side corroded columns is 10 to 35 % for columns with L/d=5 and 5 to 30% for column with L/d=2.5. As discussed in section 4.3.2, tension-side corroded columns with L/d=5 have the same flexural failure as un-corroded columns. Although un-corroded columns with L/d=2.5 fail in flexural-shear, mode of failure of tension-side corroded columns with L/d=2.5 depends on steel
reinforcing ratio. Lower steel reinforcing ratio results in flexural failure and higher steel reinforcing ratio leads the corroded column fail in flexural-shear mode. There are several factors that affect the lateral capacity of tension-side corroded columns.

**5.3.1 Effect of Corrosion Level**

As shown in Figure 5-10 through Figure 5-12, high corrosion level results in reduction of lateral capacity. Increase in corrosion level reduces lateral capacity of tension-side corroded columns uniformly, as shown in Figure 5-12. This lateral capacity reduction is just because of smaller cross-sectional area and lower yielding stress of tensile corroded bars. When corrosion level increases from 25% to 50%, tension-side corroded columns have an average of 13% reduction in lateral strength.

![Figure 5-10 Effect of corrosion level and length of corroded zone on lateral capacity of tension-side corroded columns (L/d=5)](image-url)
5.3.2 Effect of Length of Corroded Zone

As shown in Figure 5-10 and Figure 5-13, length of corroded zone has slight effect on lateral capacity of columns with L/d=5. However, Figure 5-11 and Figure 5-14 show that longer the corroded zone, the lower the lateral capacity in corroded columns with L/d=2.5 is. In columns with zero axial load, effect of length of corroded zone is negligible.
When the length of corroded zone is doubled in tension-side corroded columns, the lateral capacity is reduced by maximum 5%. As the corroded tensile bars in columns which fail in flexural mode (columns with L/d=5 and columns with L/d=2.5 and low reinforcing ratio) can develop their yielding capacity regardless of their corroded length, the lateral capacity does not change significantly due to increase in length of corroded zone. Also, initial stiffness and ductility of columns remain the same.

![Graphs showing effect of length of corroded zone on lateral capacity of tension-side corroded columns (L/d=5)](image)

*Figure 5-13 Effect of length of corroded zone on lateral capacity of tension-side corroded columns (L/d=5)*
Figure 5-14 Effect of length of corroded zone on lateral capacity of tension-side corroded columns (L/d=2.5)

5.3.3 Effect of Compressive Strength of Concrete

Figure 5-15 shows effect of $f'_c$ on lateral capacity reduction (ratio of lateral capacity of corroded column to un-corroded column) of tension-side corroded columns. Tension-side corroded columns with different $f'_c$ have slight differences in lateral capacity reduction, particularly for lower axial load ratios.
5.3.4 Effect of Steel Reinforcing Ratio ($\rho$)

Lateral capacity reduction of tension-side corroded columns is not be affected directly by differing steel reinforcing ratios. Steel reinforcing ratio could change the mode of failure of tension-side corroded columns. The more reinforcing ratio, the column tends to fail in shear failure mode.

5.3.5 Effect of Axial Load

Figure 5-16 shows the effect of axial load on lateral capacity reduction of tension-side corroded columns. The higher the axial load, the less lateral capacity reduction in tension-side
corroded columns. As mentioned above, lateral capacity reduction of tension-side corroded columns is basically due to smaller area loss and lower yielding stress of corroded bars. Therefore, their behavior follows the response of un-corroded columns. Un-corroded columns with high axial load carry more loads than columns with low axial load, because all the studied columns are designed to be in tension-zone of P-M diagram.

![Figure 5-16 Effect of axial load on lateral capacity reduction of tension-side corroded columns](image)

Obviously, axial load has more effect on lateral capacity reduction of columns with higher corrosion levels. The average lateral capacity reduction due to decrease of axial load in tension-side corroded columns with L/d=2.5 (8%) is less than columns with L/d=5 (14%).
5.3.6 Effect of Shear Span to Depth Ratio (L/d)

Figure 5-17 shows the effect of shear span to depth ratio on lateral capacity reduction of tension-side corroded columns. Columns with L/d=2.5 have less lateral capacity reduction in comparison with columns with L/d=5 due to corrosion in tension-side.

![Figure 5-17 Effect of L/d on lateral capacity reduction of tension-side corroded column](image)

5.4 All-sides Corroded Columns

Generally, lateral capacity reduction due to severe corrosion of all-sides corroded columns is 45 to 80 % for columns with L/d=5 and 35 to 70% for column with L/d=2.5. As discussed in section 4.3.3, all-sides corroded columns with L/d=5 have the same flexural failure as un-corroded columns. Although un-corroded columns with L/d=2.5 fail in flexural-shear, mode of failure of all-sides corroded columns with L/d=2.5 depends on axial load; columns subjected to lower axial load fail in flexure and columns with high axial load fail in flexural-shear mode. There are several factors that affect the lateral capacity of all-sides corroded columns.
5.4.1 Effect of Corrosion Level

As shown in Figure 5-18 and Figure 5-19, higher corrosion level causes more decrease in lateral capacity of all-sides corroded columns. Effect of corrosion level is higher when the axial load is low. Increase in corrosion level results in less cross-sectional area of corroded bars and yielding stress of tensile corroded bars plus less buckling stress of compressive corroded bars.

All these factors cause the all-side corroded columns to carry low lateral load. When corrosion level increases from 25% to 50%, the columns have an average of 25% reduction in lateral strength.

![Figure 5-18](image-url) Effect of corrosion level and length of corroded zone on lateral capacity of all-sides corroded columns (L/d=5)
5.4.2 Effect of Length of Corroded Zone

As shown in Figure 5-18 through Figure 5-21, longer corroded zone results in less lateral capacity. Reduction in the lateral capacity of columns with high axial load due to longer corroded zone is more. When the length of corroded zone is doubled in compression-side corroded columns, the lateral capacity reduced by about 10% to 40%. Effect of higher corroded length zone on lateral strength of compression-side corroded columns with higher $f_c'$ is less (about 3% to 30%). In general, columns with high axial load and longer corroded zone exhibit less ductility. The initial stiffness of all-sides corroded columns with high axial load drops suddenly because of doubled length of corroded zone.
Figure 5-20 Effect of length of corroded zone on lateral capacity of all-sides corroded columns (L/d=5)
5.4.3 Effect of Compressive Strength of Concrete

Figure 5-22 shows the effect of $f'_c$ on lateral capacity reduction (ratio of lateral capacity of corroded column to un-corroded column) for all-sides corroded columns. All-sides corroded columns with different $f'_c$ have slight difference on lateral capacity reduction, especially on lower axial load ratios.
Figure 5-22 Effect of $f'_c$ on lateral capacity reduction of all-sides corroded columns (L/d=5)

5.4.4 Effect of Steel Reinforcing Ratio ($\rho$)

Figure 5-23 shows the effect of $\rho$ on lateral capacity reduction (ratio of lateral capacity of corroded column to un-corroded column) of all-sides corroded columns with L/d=5. All-sides corroded columns with different $\rho$ have slight difference on lateral capacity reduction, especially for columns with $f'_c = 7ksi$. 
Figure 5-23 Effect of $\rho$ on lateral capacity reduction of all-sides corroded columns (L/d=5)

Figure 5-24 shows the effect of $\rho$ on lateral capacity reduction (ratio of lateral capacity of corroded column to un-corroded column) of all-sides corroded columns with L/d=2.5. In general, lateral capacity reduction in all-sides corroded columns with lower $\rho$ is more than columns with higher reinforcing ratio, when the axial load is low. Columns with higher steel reinforcing ratio have higher flexural capacity and tend to fail in shear; therefore, corrosion has less effect on these columns.

Figure 5-24 Effect of $\rho$ on lateral capacity reduction of all-sides corroded columns (L/d=2.5)
5.4.5 Effect of Axial Load

Figure 5-25 shows the effect of axial load on lateral capacity reduction of all-sides corroded columns. All-sides corroded columns with corrosion level of 25% have a slight reduction in lateral capacity when they are subjected to higher axial load. However, columns with corrosion level of 50% have a slight increase and then decrease in lateral capacity when they are subjected to higher axial load.

Figure 5-25 Effect of axial load on lateral capacity reduction of all-sides corroded columns

Un-corroded columns with high axial load have higher lateral strength than columns with low axial load, because all the studied columns are designed to be in tension-zone of P-M
diagram. When the bars are corroded, the columns with high axial load are in more critical situation as the corroded bars buckle soon and they have slight contribution to lateral load carrying capacity of columns. Therefore, their lateral capacity compared to un-corroded column reduces much. The reason which makes the all-sides corroded columns with corrosion level of 50% and zero axial load have less lateral capacity in comparison to medium axial load ratios (5% and 15%) is because: when the column is subjected to lateral displacement, the farther compression corroded bars buckle and due to a very small cross-section area of corroded tensile bars, yielding of tensile bars occur immediately after buckling. Therefore, the column develops its flexural capacity at an early stage of loading. While in the columns with medium axial load, delay in yielding of corroded tensile bars because of existence of axial load helps the column to be able to carry more loads until reaching its flexural capacity.

5.4.6 Effect of Shear Span to Depth Ratio (L/d)

Figure 5-26 shows the effect of shear span to depth ratio on lateral capacity of all-sides corroded columns. Columns with L/d=2.5 have less lateral capacity reduction due to corrosion on all-sides, generally.
5.5 Summary and Conclusion

Finite element analysis of severely corroded reinforced concrete columns is presented in details, in this chapter. Effect of different parameters has been studied on three groups of columns based on their corrosion location on the cross-section (compression-side, tension-side and all-sides corroded).

Corrosion level has the most effect on all-sides corroded columns and the least effect on compression-side corroded columns.

Length of corroded zone has the most effect on all-sides corroded columns and the least effect on tension-side (almost no effect) corroded columns.

Steel reinforcing ratio (ρ), and concrete compressive strength (f'_c) have slight impact on lateral capacity reduction of corroded columns with L/d=5. However, corroded columns with less reinforcing ratio percentage and L/d=2.5 experience higher lateral capacity reduction in comparison with same columns with higher ρ.

The higher the axial load, the higher the lateral capacity reduction in compression-side corroded columns and less lateral capacity reduction in tension-side corroded columns. Reduction in all-sides corroded columns is similar to compression-side corroded columns when corrosion level is 25%, and similar to tension-side corroded columns when corrosion level is 50%.

In general, columns with L/d=2.5 have less lateral capacity reduction due to corrosion than columns with L/d=5; because corrosion of main bars affects the flexural capacity more than shear capacity.
Table 5-1 show the lateral capacity reduction range for all studied 3 groups. Obviously, the most critical situation is for the column with all-sides corroded and L/d=5. The least effect of corrosion is seen on columns with tension-side corroded and L/d=2.5.

<table>
<thead>
<tr>
<th>L/d</th>
<th>Compression-side</th>
<th>Tension-side</th>
<th>All-sides</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>55-80%</td>
<td>65-90%</td>
<td>20-55%</td>
</tr>
<tr>
<td>2.5</td>
<td>60-85%</td>
<td>70-95%</td>
<td>30-65%</td>
</tr>
</tbody>
</table>
6 PRACTICAL MODEL TO ESTIMATE THE LATERAL CAPACITY OF SEVERELY CORRODED RC COLUMNS

6.1 Introduction

In order to estimate the lateral capacity of severely corroded reinforced concrete columns, development of a practical model is needed. A total of 308 finite element models were created, and the effects of different parameters have seen in previous chapters. In this chapter, a practical model to evaluate the lateral capacity of severely corroded RC columns is proposed.

6.2 Methodology

Columns subjected to combined compressive and lateral loads can fail in flexure, or shear. Therefore, the lateral capacity of columns can be defined as the smallest value of flexural capacity of column divided by shear span, and shear capacity of the column.

Corroded columns have three main losses:

- Losses in the mechanical performance of reinforcing bars due to the losses in their cross-sectional area and ductility,
- Losses in the effective cross-sectional area of concrete due to cracking in the cover concrete,
- Losses in the bond performance of concrete with reinforcements.

As the mechanical properties of corroded reinforcing bars, including area loss, and stress-strain relationship, and complete loss of concrete cover due to severe corrosion is well understood, lateral capacity of severely corroded RC columns can be estimated based on these properties.
The proposed equation to estimate the lateral capacity of severely corroded RC columns is as following:

\[ V_{corr} = \alpha_{corr} V_{AASHTO} \]  \hspace{1cm} (75)

\( V_{AASHTO} \), is the lateral capacity of fully-bonded corroded columns, according to AASHTO specifications. \( \alpha_{corr} \) is the lateral capacity reduction factor which includes the effect of de-bonding due to corrosion.

### 6.2.1 Lateral Capacity of Fully-bonded Corroded Columns

Lateral capacity of fully-bonded corroded columns, \( V_{AASHTO} \), can be computed according to AASHTO specifications. In order to calculate \( V_{AASHTO} \), the following considerations has been employed.

According to section 2.2.3, area loss and yielding stress of corroded bars should be calculated using the following equation:

\[ A_{corr} = A_0 (1 - 0.01CR) \]  \hspace{1cm} (76)

\[ f_{y,corr} = f_{yo}(1 - 0.005CR) \]  \hspace{1cm} (77)

Considering the residual cross-sectional area and yielding stress of corroded bars and removing the concrete cover at corroded sides, \( V_{AASHTO} \) can be calculates as follows:

\[ V_{AASHTO} = \min(V_n, \frac{M_n}{L}) \]  \hspace{1cm} (78)

Where:

\[ M_n = A_s f_s \left( d_s - \frac{a}{2} \right) - A'_s f'_s \left( d'_s - \frac{a}{2} \right) \]  \hspace{1cm} (79)
Where $A_s$ is the area of mild tensile reinforcement, $f_s$ is the stress in mild tensile steel, $d_s$ is the distance from compression face to centroid of tensile reinforcement, $a$ is the depth of equivalent rectangular stress block, $A'_s$ is the area of mild compressive reinforcement, $f'_s$ is the stress in mild compressive steel and $d'_s$ is the distance from compression face to centroid of compressive reinforcement.

And

$$V_n = V_c + V_s \tag{80}$$

$$V_c = 0.0316 \beta \sqrt{f'_c b_v d_v} \tag{81}$$

$$V_s = \frac{A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha}{s} \tag{82}$$

Where $b_v$ is the effective web width, $d_v$ is the effective shear depth (distance between resultants of tensile and compressive forces), $A_v$ is the area of transverse reinforcement, $f_y$ is yielding stress of transverse reinforcement and $\alpha$ is the angle between the transverse bars and main bars. $\beta$ and $\theta$ are the values which can be calculated according to Modified Compression Field Theory (MCFT). These values are given in AASHTO Table B5.2.1 as functions of average strain at mid-depth of cross-section $\varepsilon_x$, concrete compressive strength $f'_c$ and shear stress $v_u$.

$$\varepsilon_x = \left\{ \begin{array}{ll}
\frac{|M_u| + 0.5N_u + 0.5|V_u| (\cot \theta)}{2E_s A_s} & A_v \geq A_{v\text{min}} \\
\frac{|M_u| + 0.5N_u + 0.5|V_u| (\cot \theta)}{E_s A_s} & A_v < A_{v\text{min}}
\end{array} \right. \tag{83}$$

In order to calculate $\varepsilon_x$, $M_u$ could be calculated as the flexural capacity of the section, using equation (79). $V_u$ could be considered as the equivalent shear force of $M_u/L$; where $L$ is the
shear span, \( \theta \) needs to be assumed at first iteration. Knowing \( \varepsilon_x \) and \( \theta_u / f'_c \), the values for \( \beta \) and \( \theta \) are obtained from the table. Iteration is repeated to achieve a convergence in \( \theta \).

### 6.2.2 Lateral Capacity Reduction Factor \( \alpha_{corr} \)

Factor \( \alpha_{corr} \) reflects the effect of de-bonding and the consequences of de-bonding such as buckling of corroded bars in compression and delay in yielding of corroded bars in tension. \( \alpha_{corr} \) depends on many parameters such as compressive strength of concrete, corrosion level, length of corroded zone, steel reinforcing ratio and the most important factor, location of corrosion in column. Different equations for \( \alpha_{corr} \) are described in the following sections. It should be noted that \( \alpha_{corr} \) should not exceed 1.

### 6.3 Practical Model for Compression-side Corroded Columns

Lateral capacity of compression-side corroded columns can be estimated as follows:

\[
V_{corr} = \alpha_{corr} V_{AASHTO}
\]  \( \text{(84)} \)

Calculation of \( V_{AASHTO} \) has been described in section 6.2.1. \( \alpha_{corr} \) for compression-side corroded columns that failed in flexural mode is proposed as:

\[
\alpha_{corr} = - \left( 1 - 0.15 \frac{f'_c}{\varepsilon} \right) NR + (90 + 7 \frac{f'_c}{\varepsilon}) \varphi_1 \varphi_2 \varphi_3 \leq 1
\]  \( \text{(85)} \)

where \( f'_c \) is the compressive strength of concrete in (ksi) and \( NR \) is axial load ratio:

\[
NR = \frac{N}{A_g f'_c} \times 100
\]  \( \text{(86)} \)

\( N \) is the axial load in (kips) and \( A_g \) is the total cross-sectional area of column in (in\(^2\)).
\[ \varphi_1 = \frac{1 - 0.08 \left( \frac{H}{24} \right)}{f_c'} \]  
(87)

\[ \varphi_2 = 1 - 0.01 (\rho - 3)(1 + 0.6(4 - f'_c)) \]  
(88)

\[ \varphi_3 = 1 + 0.005 (CR - 25)(1 + 0.265(4 - f'_c)) \]  
(89)

\( H \) is the length of corroded zone in (inch), \( \rho \) is steel reinforcing ratio, \( CR \) is corrosion level and \( f'_c \) is the compressive strength of concrete in (ksi). \( \varphi_1 \varphi_2, \varphi_1 \varphi_3 \) and \( \varphi_2 \varphi_3 \) shouldn’t exceed 1.08.

Figure 6-1 Proposed \( \alpha_{corr} \) reduction factor for compression-side corroded columns with \( L/d=5 \) and \( f'_c = 4 \) ksi
Figure 6-2 Proposed $\alpha_{corr}$ reduction factor for compression-side corroded columns with $L/d=5$ and $f'_c = 7 \, ksi$

Figure 6-1 and Figure 6-2 show the proposed equation for reduction factor, $\alpha_{corr}$. It is noticeable that the lateral capacity of columns with zero axial load is much more in FEM analysis than calculations based on AASHTO. Since the tensile bars yield and then strain goes beyond the yielding point (strain hardening part), columns could carry higher lateral load.
The proposed practical model was used to compute the ultimate lateral capacity of all the cases studied using the FEA model. A comparison between the results obtained from the proposed practical model and the FEA model is shown in Figure 6-3.

### 6.4 Practical Model for Tension-side Corroded Columns

Lateral capacity of tension-side corroded columns can be estimated as follows:

\[
V_{\text{corr}} = \alpha_{\text{corr}} V_{\text{AASHTO}}
\]  

(90)

\( V_{\text{AASHTO}} \), which has been discussed in section 6.2.1, has been computed for all the cases studied using the FEA model. The results show that the lateral capacity of tension-side corroded columns in FEM analysis is much more than calculations based on AASHTO.
Since the tensile corroded bars have premature yielding and then strain goes beyond the yielding point (strain hardening part), columns could carry higher lateral load. Therefore, $\alpha_{corr}$ should be considered equal to 1 for tension-side corroded columns.

### 6.5 Practical Model for All-sides Corroded Columns

Lateral capacity of all-sides corroded columns can be estimated as follows:

$$V_{corr} = \alpha_{corr} V_{AASHTO}$$  \hfill (91)

Calculating of $V_{AASHTO}$ has been described in section 6.2.1. $\alpha_{corr}$ for all-sides corroded columns failed in flexural mode is proposed as:

$$\alpha_{corr} = - \left( 2.3 - 0.9 \frac{f'_c}{4} + 0.9(H - 24) \right) NR + \left( 106 - 8 \frac{f'_c}{4} \right) \varphi_3 \leq 1$$  \hfill (92)

where $f'_c$ is the compressive strength of concrete in (ksi), $H$ is the length of corroded zone in (inch) and $NR$ is axial load ratio:

$$NR = \frac{N}{\frac{A_g}{f'_c}} \times 100$$  \hfill (93)

$N$ is the axial load in (kips) and $A_g$ is the total cross-sectional area of column in ($in^2$).

$$\varphi_3 = 1 + 0.002(CR - 25)$$  \hfill (94)

$CR$ is corrosion level.

$\alpha_{corr}$ should be considered equal to 1 when

$$\frac{CR}{\rho} (13.74H - 1)(2.2 - .05CR + \frac{0.1}{f'_c}) \leq 4$$  \hfill (95)
Figure 6-5 Proposed $\alpha_{corr}$ reduction factor for all-sides corroded columns with L/d=5 and $f'_c = 4$ ksi

Figure 6-6 Proposed $\alpha_{corr}$ reduction factor for all-sides corroded columns with L/d=5 and $f'_c = 7$ ksi

Figure 6-5 and Figure 6-6 show the proposed equation for reduction factor, $\alpha_{corr}$. It is noticeable that the lateral capacity of some of the columns with zero axial load based on FEM, analysis is higher than calculations based on AASHTO. Since the tensile bars yield and then
strain goes beyond the yielding point (strain hardening part), columns could carry higher lateral load.

The proposed practical model was used to compute the ultimate lateral capacity of all the cases studied using the FEA model. A comparison between the results obtained from the proposed practical model and the FEA model is shown in Figure 6-7.

![Figure 6-7 Comparison of lateral capacity of all-sides corroded columns based on proposed practical model with FEA results](image)

### 6.6 Discussion on Corroded Columns with Flexural-shear or Shear Failure

Columns with L/d=2.5 have generally flexural-shear or shear failure type. Corroded columns with L/d=2.5 may fail in any failure mode including flexural, flexural-shear and shear failure. Therefore, mode of failure in columns with L/d=2.5 in most studied cases could completely be changed due to corrosion. On the other hand, predicting shear capacity of concrete elements is always difficult and all the proposed models even for un-corroded columns provided
in design codes are too conservative (Figure 6-8). The experimental data is usually scattered for shear failure mode.

![Figure 6-8 Comparison of shear capacity of experimental tests and proposed ACI equations (ACI-ASCE Committee 426, 1978)](image)

Applying the same practical proposed models discussed in previous sections, the following graphs could be obtained. The solid dots represent columns failed in flexural mode. As shown in Figure 6-9, the proposed model is too conservative to predict the lateral capacity of corroded columns with L/d=2.5, same as all proposed code models for shear mode failure. However, research to propose a more accurate practical model to predict the shear capacity of corroded columns is suggested for future studies.
Figure 6-9 Comparison of lateral capacity of corroded columns failed in shear mode based on proposed practical model with FEA results; (a) tension-side corroded columns, (b) compression-side corroded columns, (c) all-sides corroded columns

6.7 Summary and Conclusion

A practical model has been proposed in this chapter to predict the lateral capacity of severely corroded columns, when flexure dominates their mode of failure. Response of 308 finite element models have been employed to propose the practical models.
As the mechanical properties of corroded reinforcing bars including area loss and stress-strain relationship, and complete loss of concrete cover due to severe corrosion is well understood, lateral capacity of severely corroded RC columns could be estimated based on these properties. A lateral capacity reduction factor, $\alpha_{corr}$, which includes the effect of de-bonding due to corrosion has been introduced. Proposed lateral capacity reduction factor is different for corroded columns based on the location of corrosion in the cross-section. It shouldn’t exceed 1.

Comparison between the proposed model and finite element analysis shows that the proposed practical model is in good agreement with existing data for severely corroded columns.
7 SUMMARY AND CONCLUSIONS

A new methodology was developed to evaluate the current lateral strength of severely corroded RC columns, which can be adapted to existing bridge condition evaluation methods. A Finite Element Analysis (FEA) model to simulate severely corroded columns was created and verified against experimental data conducted by other researchers. After being verified against experimental data, a total of 308 Finite element models were developed to investigate several variables that affect the lateral response of corroded columns. A series of 24 in \( \times \) 24 in square column sections having different material properties were modeled as cantilevers. Location of corrosion within the cross-section (Compression-side corroded, Tension-side corroded, All-sides corroded), corrosion level (CR=25\%, 30\%, 35\%, 40\%, 45\%, 50\%), length of corroded zone along the column height (1H=24in, 2H=48in), axial load ratio \((NR = \frac{P}{f'_cA_g})\) 0\%, 5\%, 15\%, 25\%), compressive strength of concrete \((f'_c = 4ksi, 7ksi)\), steel reinforcing ratio \((\rho = 2\%, 3\%, 4\%)\) and shear span to depth ratio \((L/d=2.5, 5)\) were the variables investigated in this study. For severely corroded RC columns, stirrups were assumed to be completely deteriorated and the concrete cover spalled off. Therefore, concrete cover and stirrups were removed at corroded locations. The corroded bars were assumed to be completely un-bonded to the surrounding concrete.

Based on results obtained from the finite element analysis, a practical model was developed. The proposed practical method considers all the changes in material and geometry properties including area loss of corroded steel bars and concrete cover, bond deterioration and its consequences on corroded bars’ buckling, location of corroded zone, length of corroded zone along the column, compressive strength of concrete, reinforcing ratio of RC column section,
axial load ratio, and shear span to depth ratio. This study also provides engineers better understanding of lateral response of severely corroded RC bridge columns with detailed force-displacement diagrams based on finite element analysis.

The following conclusions are drawn from the finite element analysis of severely corroded square columns, detailed investigation on impact of different parameters on lateral response of corroded columns, and the proposed practical model:

### 7.1 General Conclusions

- Response of severely corroded RC columns to combined axial and lateral load depends on location of corroded zone. Columns corroded at tension-side, compression-side or all-sides exhibit different responses.

- The lateral capacity of severely corroded columns decreases drastically compared to un-corroded columns. Lateral capacity reduction of corroded columns is 35% to 80% for all-sides corroded, 15% to 45% for compression-side corroded and 5 to 35% for tension-side corroded columns.

- Corrosion of steel bars may alter the mode of failure of RC columns. Although un-corroded columns with low shear span to depth ratio fail in flexural-shear or shear, corroded columns could fail in flexural, flexural-shear or shear mode based on their axial load, steel reinforcing ratio and corrosion level. However, corroded columns with large shear span to depth ratio have the same flexural failure as un-corroded columns.

- Columns with higher shear span to depth ratio have higher lateral capacity reduction due to corrosion than columns with low L/d, which is primarily due to
concrete cover loss and then corrosion level and length of corroded zone. However, the effect of corrosion level and length of corroded zone on lateral capacity reduction for corroded columns with L/d=2.5 is more than corroded columns with L/d=5.

- All-sides corroded columns with L/d=5 have the most critical situation with the highest lateral capacity reduction. Tension-side corroded columns with L/d=2.5 are affected least by corrosion.
- Reinforcing ratio percentage ($\rho$), and concrete compressive strength ($f'_c$) have minimal impact on lateral capacity reduction of corroded columns with L/d=5.
- The displacement at which the column reaches its maximum lateral capacity does not change significantly due to corrosion.
- The proposed practical model is in good agreement with existing data for severely corroded columns.

As columns enter severe corrosion level:

- Corrosion level (area loss of steel bars) has the most effect on all-sides corroded columns and the least effect on compression-side corroded columns.
- Length of corroded zone has the most effect on all-sides corroded columns and the least effect on tension-side (almost no effect) corroded columns.

### 7.2 Conclusions for Compression-side Corroded Columns

- Lateral capacity reduction of compression-side corroded columns is 15% to 45%. This reduction is mainly because of loss of concrete on compression zone, buckling of compressive bars and unconfined concrete core.
- Ductility in compression-side corroded columns is less in comparison to un-corroded columns.

- Corrosion reduces the initial stiffness of compression-side corroded columns.
  Corrosion rate has insignificant impact on the initial stiffness of compression-side corroded columns.

- The more axial load, the higher lateral capacity reduction in compression-side corroded columns was observed.

7.3 Conclusions for Tension-side Corroded Columns

- Lateral capacity reduction of tension-side corroded columns is 5% to 35%. This reduction is mainly because of area loss of corroded bars, reduced yielding stress and de-bonding of corroded tensile bars.

- Ductility due to corrosion increases in tension-side corroded columns regardless of L/d ratio.

- Initial stiffness of tension-side corroded columns does not change because of corrosion.

- The more axial load, the less lateral capacity reduction in tension-side corroded columns was observed.

- Steel reinforcing ratio has a major impact on mode of failure of tension-side corroded columns.

7.4 Conclusions for All-sides Corroded Columns

- Lateral capacity reduction of all-sides corroded columns is 35% to 80%. This reduction is mainly because of loss of concrete cover on all sides, buckling of
corroded bars, un-confined concrete core, area loss of corroded bars, reduced yielding stress and de-bonding of corroded tensile bars.

- In all-sides corroded columns, ductility depends on axial load. Corroded columns with low axial load experience more ductility and corroded columns with high axial load have less ductility in comparison with un-corroded columns, regardless of L/d ratio.

- Corrosion reduces the initial stiffness of all-sides corroded columns. High corrosion levels cause less initial stiffness in all-sides corroded columns.

- Lateral capacity reduction due to different axial load ratios in all-sides corroded columns is almost the same.

### 7.5 Limitations and Recommendations for Future Study

1- The FEA models and the proposed practical model can be used by engineers in practice to estimate the lateral capacity of severely corroded RC columns.

2- Further studies are necessary to provide a practical model for estimating the lateral capacity of severely corroded RC columns that fail in shear.

3- For additional verification of the developed models and due to the limited availability of experimental data, it is recommended that further experiments be performed.

4- Further studies on the following cases can give better understanding of the behavior of corroded columns subjected to lateral loading:
   - un-symmetric corrosion on cross-section
   - corrosion along the column height at other location
• corrosion along the column height having shorter length to consider in-elastic buckling of corroded bars
• effect of compressive strength of concrete in columns with \( \frac{L}{d} = 2.5 \)
• rectangular cross-section with different height to width ratio
• circular cross-sections
APENDIX 1: LATERAL LOAD- DISPLACEMENT CURVES OF COMPRESSION-SIDE CORRODED COLUMNS

C-1H-CR25-f4p3Ld5-NR05

C: Compression-side corroded specimen
H: Length of corroded zone →1H
CR: Corrosion level (Area loss of steel bars) →25%
f: Compressive strength of concrete →4ksi
ρ: Steel reinforcing ratio →3%
Ld: Shear span to depth ratio →5
NR: Axial load ratio →05%
<table>
<thead>
<tr>
<th>UC-f4p2Ld5-NR00</th>
<th>C-1H-CR25-f4p2Ld5-NR00</th>
<th>C-1H-CR50-f4p2Ld5-NR00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δ</td>
<td>V</td>
<td>Status</td>
</tr>
<tr>
<td>0.786</td>
<td>52.13</td>
<td>Y1</td>
</tr>
<tr>
<td>2.237</td>
<td>67.83</td>
<td>C</td>
</tr>
<tr>
<td>3.166</td>
<td>69.55</td>
<td>Max</td>
</tr>
</tbody>
</table>

UC: Un-corroded specimen  
C: Compression-side corroded specimen  
Y1: Yielding of bars at 1st layer  
CR: Corrosion height  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
H: Corrosion height  
P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
CR: Area loss  
B4: Buckling of bars at 4th layer  
Y1: Yielding of bars at 1st layer  
Max: Maximum lateral capacity

**Diagrams:**
- C-1H-f4p2Ld5-NR05
- C-1H-f4p2Ld5-NR05

**Legend:**
- UC-f4p2Ld5-NR05
- C-1H-1CR25-f4p2Ld5-NR05
- C-1H-1CR50-f4p2Ld5-NR05

Specifications:
- Compressive strength of concrete:
- Steel reinforcing ratio:
- Shear span to depth ratio:
- Axial load ratio:
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
F: Compressive strength of concrete  
p: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
B4: Buckling of bars at 4th layer  
Max: Maximum lateral capacity  
C: Crushing of Concrete  
Y1: Yielding of bars at 1st layer
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<th>Lateral Capacity, $V$ (kips)</th>
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### Diagram

- **UC**: Un-corroded specimen
- **C**: Compression-side corroded specimen
- **H**: Corrosion height
- **CR**: Area loss
- **f**: Compressive strength of concrete
- **$\rho$**: Steel reinforcing ratio
- **Ld**: Shear span to depth ratio
- **NR**: Axial load ratio
- **Y**: Yielding of bars at 1st layer
- **C**: Crushing of Concrete
- **Max**: Maximum lateral capacity
- **B4**: Buckling of bars at 4th layer
UC: Un-corroded specimen
C: Compression-side corroded specimen
Y1: Yielding of bars at 1st layer
B4: Buckling of bars at 4th layer
### Table: \( \Delta V \) Status

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**Graphs:**

1. **C-2H-f4p2Ld5-NR05**
   - Lateral Capacity \( V \) vs. Displacement \( \Delta \) (in)
   - Displacement, \( \Delta \) (in)
   - Lateral Capacity, \( V \) (kip)

2. **C-2H-f4p2Ld5-NR05**
   - Lateral Capacity \( V \) vs. Displacement \( \Delta \) (in)
   - Displacement, \( \Delta \) (in)
   - Lateral Capacity, \( V \) (kip)

**Legends:**
- UC: Un-corroded specimen
- C: Compression side corroded specimen
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- H: Corrosion height
- LD: Shear span to depth ratio
- NR: Axial load ratio
- Max: Maximum lateral capacity
- B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen
C: Compression-side corroded specimen
Y1: Yielding of bars at 1st layer
CR: Crushing of Concrete
CR: Area loss
NR: Axial load ratio
B4: Buckling of bars at 4th layer

![Graph of C-2H-F4p2Ld5-NR15](image-url)

![Graph of C-2H-F4p2Ld5-NR15](image-url)
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| **C-2H-CR2S-44p2Ld5-NR25** | | | | |
| $\Delta$ | $V$ | Status | $\Delta$ | $V$ | Status |
| 1.123 | 0.00 | B4 | 1.063 | 81.41 | Y1 |
| 1.562 | 50.96 | Y1 |

| **C-2H-CR50-44p2Ld5-NR25** | | | | |
| $\Delta$ | $V$ | Status | $\Delta$ | $V$ | Status |
| 1.163 | 51.19 | Max |

UC: Un-corroded specimen  
C: Compression-side corroded specimen  
Y1: Yielding of bars at 1st layer  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
C: Crushing of Concrete  
CR: Area loss  
NR: Axial load ratio  
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
H: Corrosion height  
Ld: Shear span to depth ratio  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer  
NR: Axial load ratio  
f: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen
C: Compression-side corroded specimen
p: Steel reinforcing ratio
H: Corrosion height
Ld: Shear span to depth ratio
NR: Axial load ratio
B4: Buckling of bars at 4th layer

Displacement, $\Delta$ (in)

Lateral Capacities, $V$ (kip)
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
Y1: Yielding of bars at 1st layer  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
B4: Buckling of bars at 4th layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity
<table>
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UC: Un-corroded specimen
C: Compression-side corroded specimen
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B4: Buckling of bars at 4th layer

UC: Un-corroded specimen
C: Compression-side corroded specimen
H: Corrosion height
Ld: Shear span to depth ratio
NR: Axial load ratio
B4: Buckling of bars at 4th layer

UC-f4p3Ld5-NR05
C-2H-CR25-f4p3Ld5-NR05
C-2H-CR50-f4p3Ld5-NR05

0.170 20.82 B4 0.114 14.02 B4
0.960 78.80 Y1 1.506 62.36 Y1
1.826 98.56 C 2.107 67.79 C
1.871 98.73 Max 2.517 68.81 Max
2.592 67.17 Max
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UC: Uncorroded specimen
C: Compression-side corroded specimen
Y: Yielding of bars at 1st layer
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
NR: Axial load ratio
B: Buckling of bars at 4th layer
Max: Maximum lateral capacity

**Legend**
- UC-f4p3Ld5-NR15
- C-2H-CR25-f4p3Ld5-NR15
- C-2H-CR50-f4p3Ld5-NR15

**Graphs**
- C-2H-f4p3Ld5-NR15
- C-2H-f4p3Ld5-NR15
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer  
F: Compressive strength of concrete  
p: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
H: Corrosion height  
ρ: Steel reinforcing ratio
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UC: Un-corroded specimen
C: Compression-side corroded specimen
F: Compressive strength of concrete
p: Steel reinforcing ratio
H: Corrosion height
Ld: Shear span to depth ratio
NR: Axial load ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B4: Buckling of bars at 4th layer

Layer 1 2 3 4
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
H: Corrosion height  
f: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
H: Corrosion height  
P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Max: Maximum lateral capacity  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
B4: Buckling of bars at 4th layer  
ρ: Compressive strength of concrete  
Ld: Shear span to depth ratio  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
Max: Maximum lateral capacity
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UC: Un-corroded specimen
C: Compression-side corroded specimen
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
NR: Axial load ratio
Max: Maximum lateral capacity

UC-CR250-T7p2Ld5-NR25
C-1H-CR250-T7p2Ld5-NR25
C-1H-CR50-T7p2Ld5-NR25

UC: Un-corroded specimen
C: Compression-side corroded specimen
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
NR: Axial load ratio
Max: Maximum lateral capacity

f: Compressive strength of concrete
ρ: Steel reinforcing ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
B4: Buckling of bars at 4th layer
### Table: ΔV Status

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**Legend:**
- UC: Un-corroded specimen
- C: Compression-side corroded specimen
- H: Corrosion height
- La: Shear span to depth ratio
- NR: Axial load ratio
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity
- B4: Buckling of bars at 4th layer

**Graphs:**
- C-2H-f7p2Ld5-NR00
- C-2H-CR250-f7p2Ld5-NR00
- C-2H-CR50-f7p2Ld5-NR00

**Notes:**
- f: Compressive strength of concrete
- ρ: Steel reinforcing ratio
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UC: Un-corroded specimen  
f: Compressive strength of concrete  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of concrete  
B4: Buckling of bars at 4th layer  
H: Corrosion height  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
Max: Maximum lateral capacity
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UC: Un-corroded specimen  
F: Compressive strength of concrete  
Y1: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
H: Corrosion height  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
Max: Maximum lateral capacity  
f: Compressive strength of concrete  
$\rho$: Steel reinforcing ratio  
B4: Buckling of bars at 4th layer
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### Table: ΔV Status

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### Graph: Lateral Capacity vs. Displacement

- UC: Un-corroded specimen
- C: Compression-side corroded specimen
- H: Corrosion height
- Ld: Shear span to depth ratio
- NR: Axial load ratio
- CR: Area loss
- YT: Yielding of transverse bars
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity
- B4: Buckling of bars at 4th layer

### Graph: Lateral Capacity vs. Displacement

- UC: Un-corroded specimen
- C: Compression-side corroded specimen
- H: Corrosion height
- Ld: Shear span to depth ratio
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- YT: Yielding of transverse bars
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity
- B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen
C: Compression-side corroded specimen
H: Corrosion height
CR: Area loss
f: Compressive strength of concrete
p: Steel reinforcing ratio
Ld: Shear span to depth ratio
B4: Buckling of bars at 4th layer
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
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- UC: Un-corroded specimen
- C: Compression-side corroded specimen
- f: Compressive strength of concrete
- ρ: Steel reinforcing ratio
- Ld: Shear span to depth ratio
- NR: Axial load ratio
- H: Corrosion height
- CR: Area loss
- B4: Buckling of bars at 4th layer
- Y1: Yielding of bars at 1st layer
- Y: Yielding of transverse bars
- C: Crushing of Concrete
- Max: Maximum lateral capacity
ΔV Status  ΔV Status  ΔV status
0.225 27.35  B4  0.138 21.50  B4
0.834 76.38  Y1  1.116 64.33  Y1  1.156 76.58  C
2.675 103.26  C  2.597 78.82  C  2.856 77.63  Max
3.261 106.24  Max  2.780 79.43  Max

UC: Un-corroded specimen  F: Compressive strength of concrete
C: Compression-side corroded specimen  p: Steel reinforcing ratio
H: Corrosion height  Ld: Shear span to depth ratio
CR: Area loss  NR: Axial load ratio
Y1: Yielding of bars at 1st layer  C: Crushing of Concrete
Max: Maximum lateral capacity  B4: Buckling of bars at 4th layer

Layer  1  2  3  4

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UC: Un-corroded specimen
C: Compression-side corroded specimen
H: Corrosion height
Ld: Shear span to depth ratio
NR: Axial load ratio
CR: Area loss
B4: Buckling of bars at 4th layer
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
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### Table: ΔV Status

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### Graph: Lateral Capacity vs. Displacement

- **UC**: Un-corroded specimen
- **C**: Compression-side corroded specimen
- B: Yielding of bars at 1st layer
- Max: Maximum lateral capacity

### Graph 1: C-2H-F7p3Ld5-NR25

- **UC**: Un-corroded specimen
- **C**: Compression-side corroded specimen
- B: Yielding of bars at 1st layer
- Max: Maximum lateral capacity

### Graph 2: C-2H-F7p3Ld5-NR25

- **UC**: Un-corroded specimen
- **C**: Compression-side corroded specimen
- B: Yielding of bars at 1st layer
- Max: Maximum lateral capacity
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UC: Un-corroded specimen
C: Compression-side corroded specimen
H: Corrosion height
CR: Area loss
f: Compressive strength of concrete
ρ: Steel reinforcing ratio
Ld: Shear span to depth ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
H: Corrosion height  
CR: Area loss  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
YT: Yielding of transverse bars  
B4: Buckling of bars at 4th layer  
Y1: Yielding of bars at 1st layer  
Max: Maximum lateral capacity

UC-f4p3Ld2.5-NR05  
C-1H-CR25-f4p3Ld2.5-NR05  
C-1H-CR50-f4p3Ld2.5-NR05
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
F: Compressive strength of concrete  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
B4: Buckling of bars at 4th layer  

C: Crushing of Concrete  
H: Corrosion height  
LD: Shear span to depth ratio  
NR: Axial load ratio  
Max: Maximum lateral capacity  

<table>
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**UC-f4p3Ld2.5-NR25**

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**C-1H-CR25-f4p3Ld2.5-NR25**

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**C-1H-CRS50-f4p3Ld2.5-NR25**

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**UC-f4p3Ld2.5-NR25**

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</tr>
<tr>
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<td>Y1</td>
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<tr>
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<td>C</td>
</tr>
<tr>
<td>0.670</td>
<td>206.99</td>
<td>Max</td>
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UC: Un-corroded specimen  
f: Compressive strength of concrete  
C: Compression-side corroded specimen  
p: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Max: Maximum lateral capacity  
Yielding of bars at 1st layer  
Yielding of bars at 4th layer  
Crushing of Concrete  
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
H: Corrosion height  
CR: Area loss  
F: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity
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<thead>
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**Legend:**
- UC: Un-corroded specimen
- C: Compression-side corroded specimen
- H: Corrosion height
- CR: Area loss
- f: Compressive strength of concrete
- p: Steel reinforcing ratio
- Ld: Shear span to depth ratio
- NR: Axial load ratio
- YT: Yielding of transverse bars
- Y1: Yielding of bars at 1st layer
- B4: Buckling of bars at 4th layer
- Max: Maximum lateral capacity
<table>
<thead>
<tr>
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UC: Un-corroded specimen
f: Compressive strength of concrete
C: Compression-side corroded specimen
H: Corrosion height
p: Steel reinforcing ratio
Ld: Shear span to depth ratio
NR: Axial load ratio
YT: Yielding of transverse bars
B4: Buckling of bars at 4th layer
Y1: Yielding of bars at 1st layer
CR: Area loss
Max: Maximum lateral capacity
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**UC:** Un-corroded specimen  
**C:** Compression-side corroded specimen  
**f:** Compressive strength of concrete  
**ρ:** Steel reinforcing ratio  
**Ld:** Shear span to depth ratio  
**NR:** Axial load ratio  
**YT:** Yielding of transverse bars  
**Y1:** Yielding of bars at 1st layer  
**Max:** Maximum lateral capacity  

**UC-f4p3Ld2.5-NR25**  
**C-2H-CR25-f4p3Ld2.5-NR25**  
**C-2H-CR50-f4p3Ld2.5-NR25**

UC: Un-corroded specimen  
C: Compression-side corroded specimen  
F: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
Max: Maximum lateral capacity
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<td>( d ) f status</td>
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<td>0.733 188.82 C</td>
<td>0.840 190.67 Max</td>
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<tr>
<td>0.184 84.56 B4</td>
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UC: Un-corroded specimen  
\( f \): Compressive strength of concrete  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
B4: Buckling of bars at 4th layer  
C: Crushing of Concrete  
H: Corrosion height  
\( \rho \): Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
CR: Area loss  
Max: Maximum lateral capacity
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<td>0.911</td>
<td>170.36</td>
<td>Max</td>
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UC: Un-corroded specimen
C: Compression-side corroded specimen
H: Corrosion height
CR: Area loss
f: Compressive strength of concrete
ρ: Steel reinforcing ratio
Ld: Shear span to depth ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

YT: Yielding of transverse bars
B4: Buckling of bars at 4th layer
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UC: Un-corroded specimen  
C: Compression-side corroded specimen  
H: Corrosion height  
f: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
l: Shear span to depth ratio  
NR: Axial load ratio  
Y1: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer  
YT: Yielding of transverse bars
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UC: Un-corroded specimen
C: Compression-side corroded specimen
CR: Corrosion height
f: Compressive strength of concrete
\(\rho\): Steel reinforcing ratio
Ld: Shear span to depth ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B4: Buckling of bars at 4th layer

Layer 1 2 3 4

Lateral Capacity, \(V\) (kip)
Displacement, \(\Delta\) (in)
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<td>Lateral Capacity, V (kip)</td>
<td>Displacement, Δ (in)</td>
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</table>

UC: Un-corroded specimen  
f: Compressive strength of concrete  
YT: Yielding of transverse bars  
B4: Buckling of bars at 4th layer  
C: Compression-side corroded specimen  
p: Steel reinforcing ratio  
Y1: Yielding of bars at 1st layer  
H: Corrosion height  
C: Crushing of Concrete  
CR: Area loss  
NR: Axial load ratio  
Max: Maximum lateral capacity
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UC: Un-corroded specimen  
f: Compressive strength of concrete  
Y1: Yielding of transverse bars  
C: Yielding of bars at 1st layer  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Max: Maximum lateral capacity  

UC: Un-corroded specimen  
f: Compressive strength of concrete  
Y1: Yielding of transverse bars  
C: Yielding of bars at 1st layer  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Max: Maximum lateral capacity
APENDIX 2: LATERAL LOAD- DISPLACEMENT CURVES OF TENSION-SIDE CORRODED COLUMNS

T-1H-CR25-f4p3Ld5-NR05

T: Tension-side corroded specimen
H: Length of corroded zone → 1H
CR: Corrosion level (Area loss of steel bars) → 25%
f: Compressive strength of concrete → 4ksi
ρ: Steel reinforcing ratio → 3%
Ld: Shear span to depth ratio → 5
NR: Axial load ratio → 05%
<table>
<thead>
<tr>
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<td>2.255</td>
<td>72.83</td>
<td>Max</td>
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UC: Un-corroded specimen
f: Compressive strength of concrete
Y1: Yielding of bars at 1st layer
T: Tension-side corroded specimen
p: Steel reinforcing ratio
H: Corrosion height
Ld: Shear span to depth ratio
C: Crushing of Concrete
CR: Area loss
ρ: Steel reinforcing ratio
NR: Axial load ratio
L: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer
<table>
<thead>
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
ρ: Steel reinforcing ratio  
2Ld: Shear span to depth ratio  
NR: Axial load ratio  
Y: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  

B: Buckling of bars at 1st layer  
S: Straightening of buckled bars at 1st layer
<table>
<thead>
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UC: Un-corroded specimen  
f: Compressive strength of concrete  
T: Tension side corroded specimen  
p: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
B1: Buckling of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
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Max: Maximum lateral capacity  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
1: Straightening of buckled bars at 1st layer  
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UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
f: Compressive strength of concrete
p: Steel reinforcing ratio
Ld: Shear span to depth ratio
C: Crushing of Concrete
Max: Maximum lateral capacity
B1: Buckling of bars at 1st layer
S1: Straightening of buckled bars at 1st layer
Lateral Capacity, \( V \) (kip)
Displacement, \( \Delta \) (in)
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<td>0.089</td>
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<td>1.063</td>
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
H: Corrosion height  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
B1: Buckling of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer  
Max: Maximum lateral capacity  
C: Crushing of Concrete
<table>
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<th>T-1H-CR25-f4p3Ld5-NR00</th>
<th>T-1H-CR50-f4p3Ld5-NR00</th>
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<tr>
<td>C Max</td>
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**Legend:**
- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- $\rho$: Steel reinforcing ratio
- Ld: Shear span to depth ratio
- CR: Area loss
- NR: Axial load ratio
- Y1: Yielding of bars at 1st layer
- Max: Maximum lateral capacity
- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- $\rho$: Steel reinforcing ratio
- Ld: Shear span to depth ratio
- CR: Area loss
- NR: Axial load ratio
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity
- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- $\rho$: Steel reinforcing ratio
- Ld: Shear span to depth ratio
- CR: Area loss
- NR: Axial load ratio
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity

**Graphs:**
- T-1H-f4p3Ld5-NR00 chart showing lateral capacity vs. displacement.
- T-1H-f4p3Ld5-NR00 chart with multiple lines indicating different conditions.

**Displacement, $\Delta$ (in):**
- Lateral Capacity, $V$ (kip)
<table>
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<tr>
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<th>T-1H-CR25-f4p3Ld5-NR05</th>
<th>T-1H-CR50-f4p3Ld5-NR05</th>
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</thead>
<tbody>
<tr>
<td>( \Delta )</td>
<td>( V )</td>
<td>Status</td>
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<tr>
<td>0.960</td>
<td>78.80</td>
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</tr>
<tr>
<td>1.826</td>
<td>98.56</td>
<td>C</td>
</tr>
<tr>
<td>1.871</td>
<td>98.73</td>
<td>Max</td>
</tr>
</tbody>
</table>

**UC:** Un-corroded specimen  
**T:** Tension-side corroded specimen  
**p:** Steel reinforcing ratio  
**Ld:** Shear span to depth ratio  
**CR:** Area loss  
**NR:** Axial load ratio  
**Y1:** Yielding of bars at 1st layer  
**C:** Crushing of Concrete  
**Max:** Maximum lateral capacity  
**S1:** Straightening of buckled bars at 1st layer  

**La yer**  
1 2 3 4
<table>
<thead>
<tr>
<th>Δ</th>
<th>V</th>
<th>Status</th>
<th>Δ</th>
<th>V</th>
<th>Status</th>
<th>Δ</th>
<th>V</th>
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<tr>
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<td>Max</td>
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</table>

UC: Un-corroded specimen
T: Tension-side corroded specimen
Y: Yielding of bars at 1st layer
S: Straightening of buckled bars at 1st layer
R: Crushing of Concrete
H: Corrosion height
L: Shear span to depth ratio
CR: Area loss
CR: Axial load ratio

UC-f4p3Ld5-NR15
T-1H-CR25-f4p3Ld5-NR15
T-1H-CR50-f4p3Ld5-NR15

Layer 1 2 3 4

Displacement, Δ (in)
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</table>

UC: Un-corroded specimen
f: Compressive strength of concrete
T: Tension side corroded specimen
p: Steel reinforcing ratio
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
NR: Axial load ratio
Y1: Yielding of bars at 1st layer
B1: Buckling of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
S1: Straigthening of buckled bars at 1st layer
## UC-f4p3Ld5-NR00 vs T-2H-CR25-f4p3Ld5-NR00 vs T-2H-CR50-f4p3Ld5-NR00

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<td>3.749</td>
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</table>

UC: Un-corroded specimen  
T: Tension-side corroded specimen  
H: Corrosion height  
CR: Area loss  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
NR: Axial load ratio  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
B1: Buckling of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
f: Compressive strength of concrete
v: Steel reinforcing ratio
Ld: Shear span to depth ratio
C: Crushing of Concrete
Max: Maximum lateral capacity
Y1: Yielding of bars at 1st layer
B1: Buckling of bars at 1st layer
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
C: Corrosion height  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
B1: Buckling of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer

Layer 1 2 3 4

T-2H-f4p3Ld5-NR15

T-1H-f4p3Ld5-NR15

UC: Un-corroded specimen  
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P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
C: Corrosion height  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
B1: Buckling of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
C: Crushing of concrete  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
Max: Maximum lateral capacity  
B1: Buckling of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer
<table>
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<th>Status</th>
<th>V</th>
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<th>V</th>
<th>Δ</th>
<th>Status</th>
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UC: Un-corroded specimen
f: Compressive strength of concrete
Y1: Yielding of bars at 1st layer
CR: Area loss
B1: Buckling of bars at 1st layer
S1: Straightening of buckled bars at 1st layer
ρ: Steel reinforcing ratio
Ld: Shear span to depth ratio
C: Crushing of Concrete
Max: Maximum lateral capacity
NR: Axial load ratio
V: Lateral capacity (kip)
Δ: Displacement (in)
<table>
<thead>
<tr>
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<th>T-1H-CR50-F7p2Ld5-NR05</th>
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<td>Δ   V Status</td>
<td>Δ   V Status</td>
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</tr>
<tr>
<td>0.766 67.24 Y1</td>
<td>0.695 59.25 Y1</td>
<td>0.466 44.50 Y1</td>
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<td>2.497 85.89 C</td>
<td>3.052 73.25 C</td>
<td>3.274 62.33 C</td>
</tr>
<tr>
<td>2.709 87.23 Max</td>
<td>3.572 74.49 Max</td>
<td>3.770 63.10 Max</td>
</tr>
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</table>

UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
NR: Axial load ratio
Max: Maximum lateral capacity

UC-F7p2Ld5-NR05
T-1H-CR25-F7p2Ld5-NR05
T-1H-CR50-F7p2Ld5-NR05

UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height

UC-F7p2Ld5-NR05
T-1H-CR25-F7p2Ld5-NR05
T-1H-CR50-F7p2Ld5-NR05

UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height

UC-F7p2Ld5-NR05
T-1H-CR25-F7p2Ld5-NR05
T-1H-CR50-F7p2Ld5-NR05
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<th>T-1H-CR50-f7p2Ld5-NR15</th>
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<td>Status</td>
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<td>1.935</td>
<td>107.87</td>
<td>Max</td>
<td>1.781</td>
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UC: Uncorroded specimen
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of concrete
Max: Maximum lateral capacity
Ld: Shear span to depth ratio
B1: Buckling of bars at 1st layer
S1: Straightening of buckled bars at 1st layer

UC-f7p2Ld5-NR15: Black line
T-1H-CR2S-f7p2Ld5-NR15: Blue line
T-1H-CR50-f7p2Ld5-NR15: Red line
<table>
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<td>1.633</td>
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
P: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity

UC-f7p2Ld5-NR25  
T-1H-CR25-f7p2Ld5-NR25  
T-1H-CR50-f7p2Ld5-NR25

Displacement, Δ (in)  
Lateral Capacity, V (kips)
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<td>4.669</td>
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
H: Corrosion height  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
CR: Area loss  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B1: Buckling of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer
Table:

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UC: Un-corroded specimen
T: Tension-side corroded specimen
p: Steel reinforcing ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

UC-F7p2Ld5-NR05
T-2H-CR25-F7p2Ld5-NR05
T-2H-CR50-F7p2Ld5-NR05

UC: Un-corroded specimen
T: Tension-side corroded specimen
p: Steel reinforcing ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

Diagram:

UC-F7p2Ld5-NR05
T-2H-CR25-F7p2Ld5-NR05
T-2H-CR50-F7p2Ld5-NR05

UC: Un-corroded specimen
T: Tension-side corroded specimen
p: Steel reinforcing ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

UC-F7p2Ld5-NR05
T-2H-CR25-F7p2Ld5-NR05
T-2H-CR50-F7p2Ld5-NR05

UC: Un-corroded specimen
T: Tension-side corroded specimen
p: Steel reinforcing ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
p: Steel reinforcing ratio  
Δd: Shear span to depth ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B1: Buckling of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer  

UC-F7p2Ld5-NR15  
T-2H-CR25-F7p2Ld5-NR15  
T-2H-CR50-F7p2Ld5-NR15
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<td>YT</td>
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<td>Y1</td>
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**UC-17p2Ld5-NR25**

**T-2H-CR25-17p2Ld5-NR25**

**T-2H-CR50-17p2Ld5-NR25**

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La: Lateral Capacity (kip)  
Δ: Displacement (in)  
UC: Un-corroded specimen  
T: Tension-side corroded specimen  
H: Corrosion height  
CR: Area loss  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
S1: Straihtening of buckled bars at 1st layer  
Max: Maximum lateral capacity  
V: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
B1: Buckling of bars at 1st layer  

UC: Un-corroded specimen  
T: Tension-side corroded specimen  
H: Corrosion height  
CR: Area loss  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
S1: Straightening of buckled bars at 1st layer  
Max: Maximum lateral capacity
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UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B1: Buckling of bars at 1st layer
S1: Straightening of buckled bars at 1st layer
Ld: Shear span to depth ratio
ρ: Steel reinforcing ratio
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<td>Layer 2</td>
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UC: Un-corroded specimen
T: Tension-side corroded specimen
p: Steel reinforcing ratio
Y: Yielding of transverse bars
V: Yielding of bars at 1st layer
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
ρ: Axial load ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

B1: Buckling of bars at 1st layer
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
T: Tension-side corroded specimen
p: Steel reinforcing ratio
Y: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

UC-f7p3Ld5-NR15: UC-steel ratio 3, Ld depth 5, NR axial load ratio 15
T-1H-CR25-f7p3Ld5-NR15: Tension side corroded with steel ratio 3, Ld depth 5, NR axial load ratio 15
T-1H-CR50-f7p3Ld5-NR15: Tension side corroded with steel ratio 3, Ld depth 5, NR axial load ratio 50
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
H: Corrosion height  
CR: Area loss  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
T: Tension-side corroded specimen
ρ: Steel reinforcing ratio
Y: Yielding of transverse bars
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
R: Axial load ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B: Buckling of bars at 1st layer
S: Straightening of buckled bars at 1st layer
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**Legend:**
- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- p: Steel reinforcing ratio
- Y1: Yielding of bars at 1st layer
- B1: Buckling of bars at 1st layer
- S1: Straightening of buckled bars at 1st layer
- H: Corrosion height
- Ld: Shear span to depth ratio
- CR: Area loss
- NR: Axial load ratio
- Max: Maximum lateral capacity
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UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
ρ: Steel reinforcing ratio
Ld: Shear span to depth ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
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**Legend:**
- UC: Un-corroded specimen
- T: Tension side corroded specimen
- p: Steel reinforcing ratio
- Ld: Shear span to depth ratio
- NR: Axial load ratio
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity
- CR: Area loss
- S1: Straightening of buckled bars at 1st layer

**Diagram Description:**
- The graphs show the lateral capacity and displacement for different conditions, indicating how the capacity varies with displacement under different load scenarios.
- The labels on the axes represent the variables being measured, such as lateral capacity (V) in kips and displacement (δ) in inches.
- The graphs are color-coded to differentiate between various conditions, with each condition represented by a distinct line.

**Formulas:**
1. **UC** = Un-corroded specimen
2. **T** = Tension side corroded specimen
3. **p** = Steel reinforcing ratio
4. **Ld** = Shear span to depth ratio
5. **NR** = Axial load ratio
6. **Y1** = Yielding of bars at 1st layer
7. **C** = Crushing of Concrete
8. **Max** = Maximum lateral capacity
9. **CR** = Area loss
10. **S1** = Straightening of buckled bars at 1st layer
### Table

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<th>Status</th>
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**UC:** Un-corroded specimen  
**T:** Tension-side corroded specimen  
**H:** Corrosion height  
**CR:** Area loss  
**f:** Compressive strength of concrete  
**p:** Steel reinforcing ratio  
**Ld:** Shear span to depth ratio  
**C:** Crushing of Concrete  
**NR:** Axial load ratio  
**Y1:** Yielding of bars at 1st layer  
**B1:** Buckling of bars at 1st layer  
**C1:** Straightening of buckled bars at 1st layer  
**La:** Layer 1  
**Ly:** Layer 2  
**Lz:** Layer 3  
**Lw:** Layer 4
<table>
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<td>C</td>
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**Legend:**
- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- H: Corrosion height
- CR: Area loss
- NR: Axial load ratio
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity
- S1: Straightening of buckled bars at 1st layer
- 1: 2: 3: 4: Layer
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<td>173.96 Y1</td>
<td>0.286 146.78 Y1</td>
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<td>0.529</td>
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<td>0.726 156.32 C</td>
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<td>0.767</td>
<td>193.66 Max</td>
<td>0.798 180.93 Max</td>
<td>0.843 157.38 Max</td>
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
C: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
CR: Corrosion height  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer  
B1: Buckling of bars at 1st layer  
La: Lateral capacity (kip)  
Displacement, Δ (in)  
Max: Maximum lateral capacity  
0.423 173.96 Y1 0.286 146.78 Y1 0.203 114.02 Y1  
0.529 186.25 C 0.654 177.72 C 0.726 156.32 C  
0.767 193.66 Max 0.798 180.93 Max 0.843 157.38 Max
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<td>0.335</td>
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<td>0.244</td>
<td>142.41</td>
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<tr>
<td>0.483</td>
<td>199.11</td>
<td>C</td>
<td>0.490</td>
<td>189.97</td>
<td>C</td>
<td>0.524</td>
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<tr>
<td>0.670</td>
<td>206.99</td>
<td>Max</td>
<td>0.620</td>
<td>192.33</td>
<td>Max</td>
<td>0.734</td>
<td>177.69</td>
<td>Max</td>
</tr>
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UC: Un-corroded specimen  
f: Compressive strength of concrete  
Y1: Yielding of bars at 1st layer  
B1: Buckling of bars at 1st layer  
T1: Tension-side corroded specimen  
p: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Y: Yielding of bars  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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<th>V</th>
<th>Status</th>
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<td>125.27</td>
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<td>146.39</td>
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![Graph](image-url)

**Graph Labels:**
- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- F: Compressive strength of concrete
- Y: Yielding of bars at 1st layer
- B: Buckling of bars at 1st layer
- L: Shear span to depth ratio
- C: Corrosion height
- CR: Area loss
- NR: Axial load ratio
- Δ: Maximum lateral capacity
- Max: Maximum lateral capacity
<table>
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<td>Δ V Status</td>
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<td>0.269 113.54 Y1</td>
<td>0.191 85.61 Y1</td>
<td>0.651 172.83 C</td>
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UC: Un-corroded specimen
T: Tension-side corroded specimen
f: Compressive strength of concrete
p: Steel reinforcing ratio
H: Corrosion height
Ld: Shear span to depth ratio
C: Crushing of Concrete
NR: Axial load ratio
S1: Straightening of buckled bars at 1st layer
S: Buckling of bars at 1st layer
B1: Buckling of bars at 1st layer
Y1: Yielding of bars at 1st layer
Max: Maximum lateral capacity

UC-F4p3Ld2.5-NR05
T-2H-CR25-F4p3Ld2.5-NR05
T-2H-CR50-F4p3Ld2.5-NR05
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<td>Y1</td>
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UC: Un-corroded specimen
T: Tension-side corroded specimen
F: Compressive strength of concrete
Y1: Yielding of bars at 1st layer
H: Corrosion height
Ld: Shear span to depth ratio
C: Crushing of Concrete
Max: Maximum lateral capacity
NR: Axial load ratio
S1: Straightening of buckled bars at 1st layer
<table>
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<td>0.455 196.73 Y1</td>
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<td>0.483 199.11 C</td>
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<td>0.670 206.99 Max</td>
<td>0.640 187.81 Max</td>
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UC: Un-corroded specimen
f: Compressive strength of concrete
Y: Yielding of bars at 1st layer
B1: Buckling of bars at 1st layer
S1: Straightening of buckled bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
NR: Axial load ratio
Ld: Shear span to depth ratio
<table>
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<tr>
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<td>Δ V Status</td>
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<td>0.289 134.45 Y1</td>
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<tr>
<td>0.733 188.82 C</td>
<td>0.835 175.40 C</td>
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<tr>
<td>0.840 190.67 Max</td>
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UC: Un-corroded specimen
t: Tensile side corroded specimen
f: Compressive strength of concrete
ρ: Steel reinforcing ratio
Ld: Shear span to depth ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

Layer 1 2 3 4
<table>
<thead>
<tr>
<th>Layer</th>
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<th>2</th>
<th>3</th>
<th>4</th>
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<table>
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<td>0.771 184.67 Max</td>
<td>1.381 164.50 Max</td>
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**Legend:**
- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- F: Compressive strength of concrete
- YT: Yielding of transverse bars
- Y1: Yielding of bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity

**Graphs:**
- T-1H-f4p4Ld2.5-NR05

Displacement, Δ (in)
Lateral Capacity, V (kip)

- UC-f4p4Ld2.5-NR05
- T-1H-CR25-f4p4Ld2.5-NR05
- T-1H-CR50-f4p4Ld2.5-NR05
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<th>T-1H-CR25-f4p4Ld2.5-NR15</th>
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<td>Δ V Status</td>
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<td>0.577 203.85 YT</td>
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<td>2</td>
<td>0.527 199.61 Y1</td>
<td>0.292 164.28 Y1</td>
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<td>3</td>
<td>0.655 207.10 C</td>
<td>0.682 201.24 C</td>
<td>0.785 181.68 C</td>
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<td>4</td>
<td>0.688 207.96 Max</td>
<td>0.697 201.40 Max</td>
<td>1.288 183.51 Max</td>
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UC: Un-corroded specimen  
T: Tension-side corroded specimen  
P: Steel reinforcing ratio  
H: Corrosion height  
CR: Area loss  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity
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<td>C: 0.588 220.08 C</td>
<td>C: 0.548 216.31 C</td>
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- UC: Un-corroded specimen
- T: Tension-side corroded specimen
- H: Corrosion height
- Ld: Shear span to depth ratio
- NR: Axial load ratio
- CR: Area loss
- C: Crushing of Concrete
- Max: Maximum lateral capacity

---

**Graph: T-1H-f4p4Ld2.5-NR25**

- Lateral Capacity, $V$ (kips)
- Displacement, $\Delta$ (in)

---

**Graph: T-1H-f4p4Ld2.5-NR25**

- Lateral Capacity, $V$ (kips)
- Displacement, $\Delta$ (in)
<table>
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<tr>
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![Graph showing lateral capacity vs. displacement for different specimens.](image-url)
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<td>C</td>
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<tr>
<td>0.828</td>
<td>194.27</td>
<td>Max</td>
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UC: Un-corroded specimen
T: Tension-side corroded specimen
H: Corrosion height
CR: Area loss
\( f \): Compressive strength of concrete
\( \rho \): Steel reinforcing ratio
\( Ld \): Shear span to depth ratio
\( NR \): Axial load ratio
Max: Maximum lateral capacity

Layer 1 2 3 4

UC-f4p4Ld2.5-NR05
T-2H-CR25-f4p4Ld2.5-NR05
T-2H-CR50-f4p4Ld2.5-NR05
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<tr>
<td>0.527 199.61 Y1</td>
<td>0.323 161.15 Y1</td>
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<td>0.655 207.10 C</td>
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<td>0.688 207.96 Max</td>
<td>0.820 196.97 Max</td>
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UC: Un-corroded specimen
T: Tension-side corroded specimen
f: Compressive strength of concrete
p: Steel reinforcing ratio
H: Corrosion height
Ld: Shear span to depth ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
<table>
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<th>Values</th>
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<td>UC: Un-corroded specimen</td>
<td>T: Tension-side corroded specimen</td>
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<tr>
<td>2</td>
<td>H: Corrosion height</td>
<td>p: Steel reinforcing ratio</td>
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<tr>
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<td>Y: Yielding of bars at 1st layer</td>
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<td>C: Crushing of Concrete</td>
<td>S: Straightening of buckled bars at 1st layer</td>
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<td>CR: Area loss</td>
<td>NR: Axial load ratio</td>
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<td>Max: Maximum lateral capacity</td>
<td>B: Buckling of bars at 1st layer</td>
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<td>Y: Yielding of transverse bars</td>
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<tr>
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<td>T: Tension-side corroded specimen</td>
<td>Y: Yielding of bars at 1st layer</td>
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<td>p: Steel reinforcing ratio</td>
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<td>Y: Yielding of bars at 1st layer</td>
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<td>C: Crushing of Concrete</td>
<td>S: Straightening of buckled bars at 1st layer</td>
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<td>UC: Un-corroded specimen</td>
<td>Y: Yielding of transverse bars</td>
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<td>T: Tension-side corroded specimen</td>
<td>Y: Yielding of bars at 1st layer</td>
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<td>C: Crushing of Concrete</td>
<td>S: Straightening of buckled bars at 1st layer</td>
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UC: Un-corroded specimen  T: Tension-side corroded specimen  H: Corrosion height  p: Steel reinforcing ratio  Ld: Shear span to depth ratio  Y: Yielding of bars at 1st layer  C: Crushing of Concrete  S: Straightening of buckled bars at 1st layer  B: Buckling of bars at 1st layer  Y: Yielding of transverse bars  S: Straightening of buckled bars at 1st layer  C: Crushing of Concrete  Max: Maximum lateral capacity
APENDIX 3: LATERAL LOAD- DISPLACEMENT CURVES OF ALL-SIDES CORRODED COLUMNS

A-1H-CR25-f4ρ3Ld5-NR05

A: All-sides corroded specimen
H: Length of corroded zone → 1H
CR: Corrosion level (Area loss of steel bars) → 25%
f: Compressive strength of concrete → 4ksi
ρ: Steel reinforcing ratio → 3%
Ld: Shear span to depth ratio → 5
NR: Axial load ratio → 05%
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<thead>
<tr>
<th>Layer</th>
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<td>0.489 18.75 Y1</td>
<td>2.237 67.83 C</td>
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<td>0.489 18.75 Y1</td>
<td>3.166 69.55 Max</td>
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UC: Un corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
C: Crushing of Concrete  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
Y1: Yielding of bars at 1st layer  
S1: Straightening of buckled bars at 1st layer
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<td>1.185 39.64 Max</td>
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
B2: Buckling of bars at 2nd layer
Y1: Yielding of bars at 1st layer
B1: Buckling of bars at 1st layer
Y2: Yielding of bars at 2nd layer
C: Crushing of concrete
Max: Maximum lateral capacity

UC-f4p2Ld5-NR15
A-1H-CR25-f4p2Ld5-NR15
A-1H-CR50-f4p2Ld5-NR15

Displacement, Δ (in)
Lateral Capacity, V (kips)
| Layer | 1 | 2 | 3 | 4 |

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UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
CR: Area loss  
NR: Axial load ratio  
Ld: Shear span to depth ratio  
ρ: Steel reinforcing ratio  
f: Compressive strength of concrete  
Y1: Yielding of bars at 1st layer  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
F: Compressive strength of concrete
Ld: Shear span to depth ratio
NR: Axial load ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
S1: Straightening of buckled bars at 1st layer
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<td>Δ V Status</td>
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UC: Un-corroded specimen  
f: Compressive strength of concrete  
Y1: Yielding of bars at 1st layer  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
Max: Maximum lateral capacity  

UC-f4p2Ld5-NR05  
A2H-CR25-f4p2Ld5-NR05  
A2H-CR50-f4p2Ld5-NR05

UC: Un-corroded specimen  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
C: Crushing of Concrete  
H: Corrosion height  
LR: Shear span to depth ratio  
NR: Axial load ratio  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
### Table 1: Lateral Capacity and Displacement Data

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### Diagram 1: Lateral Capacity vs. Displacement Graph

- **UC-f4p2Ld5-NR15**
- **A2H-CR25-f4p2Ld5-NR15**
- **A2H-CR50-f4p2Ld5-NR15**

- UC: Un-corroded specimen
- f: Compressive strength of concrete
- p: Steel reinforcing ratio
- H: Corrosion height
- Ld: Shear span to depth ratio
- CR: Area loss
- NR: Axial load ratio
- Y1: Yielding of bars at 1st layer
- S1: Straightening of buckled bars at 1st layer
- C: Crushing of Concrete
- Max: Maximum lateral capacity
- B4: Buckling of bars at 4th layer
- B3: Buckling of bars at 3rd layer
- B2: Buckling of bars at 2nd layer
- B1: Buckling of bars at 1st layer
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<td>1.263</td>
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**Graphs:**

1. **A-2H-f4p2Ld5-NR25**
   - UC: Un-corroded specimen
   - A: All-sides corroded specimen
   - H: Corrosion height
   - Ld: Shear span to depth ratio
   - NR: Axial load ratio
   - CR: Area loss ratio
   - Max: Maximum lateral capacity
   - UC: Un-corroded specimen
   - S1: Straightening of buckled bars at 1st layer
   - Y1: Yielding of bars at 1st layer
   - C: Crushing of Concrete

2. **A-2H-f4p2Ld5-NR25**
   - UC: Un-corroded specimen
   - A: All-sides corroded specimen
   - H: Corrosion height
   - Ld: Shear span to depth ratio
   - NR: Axial load ratio
   - CR: Area loss ratio
   - Max: Maximum lateral capacity
   - UC: Un-corroded specimen
   - S1: Straightening of buckled bars at 1st layer
   - Y1: Yielding of bars at 1st layer
   - C: Crushing of Concrete
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<td>93.30 C</td>
<td>1.668 49.49 C</td>
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**UC**: Un-corroded specimen

- **A**: All-sides corroded specimen
- **f**: Compressive strength of concrete
- **H**: Corrosion height
- **Ld**: Shear span to depth ratio
- **CR**: Area loss
- **Y1**: Yielding of bars at 1st layer
- **B4**: Buckling of bars at 4th layer
- **B3**: Buckling of bars at 3rd layer
- **C**: Crushing of Concrete
- **Max**: Maximum lateral capacity
- **NR**: Axial load ratio
- **S1**: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
B: Buckling of bars
S: Straightening of buckled bars
R: Shear span to depth ratio
NR: Axial load ratio
Max: Maximum lateral capacity

UC-F4p3Ld5-NR05
A-1H-CR25-F4p3Ld5-NR05
A-1H-CR50-F4p3Ld5-NR05
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UC: Un-corroded specimen  
A: All-sides corroded specimen  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
La: Straightening of buckled bars at 1st layer  
NR: Axial load ratio  
CR: Area loss  
Ld: Shear span to depth ratio  
H: Corrosion height  
ρ: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
CR: Area loss  
Ld: Shear span to depth ratio  
H: Corrosion height  
ρ: Steel reinforcing ratio  
UC: Un-corroded specimen  
A: All-sides corroded specimen  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
La: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen  
A: All-sides corroded specimen  
P: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
B3: Buckling of bars at 3rd layer  
B4: Buckling of bars at 4th layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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<td>94.66</td>
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UC: Un-corroded specimen
A: All-sides corroded specimen
B1: Yielding of bars at 1st layer
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
C: Crushing of Concrete
Max: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer
Ld: Shear span to depth ratio
CR: Area loss
NR: Axial load ratio
Ω: Unfished specimen
ρ: Corrosion height
f: Compressive strength of concrete
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
Ld: Shear span to depth ratio
Max: Maximum lateral capacity
NR: Axial load ratio
Y1: Yielding of bars at 1st layer
B3: Buckling of bars at 3rd layer
B4: Buckling of bars at 4th layer
C: Crushing of Concrete
S1: Straightening of buckled bars at 1st layer

Displacement, Δ (in)

Lateral Capacity, V (kips)
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- UC: Un-corroded specimen
- A: All-sides corroded specimen
- H: Corrosion height
- Ld: Shear span to depth ratio
- NR: Axial load ratio
- Max: Maximum lateral capacity
- S1: Straightening of buckled bars at 1st layer
- B4: Buckling of bars at 4th layer
- B3: Buckling of bars at 3rd layer
- C: Crushing of Concrete
- Y1: Yielding of bars at 1st layer
- f: Compressive strength of concrete
- p: Steel reinforcing ratio
- ρ: Steel reinforcing ratio
- Ld: Shear span to depth ratio
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**Legend:**
- UC: Un-corroded specimen
- A: All-sides corroded specimen
- H: Corrosion height
- CR: Area loss
- Ld: Shear span to depth ratio
- NR: Axial load ratio
- Max: Maximum lateral capacity
- UC: Un-corroded specimen
- S1: Straightening of buckled bars at 1st layer
- B4: Buckling of bars at 4th layer
- B3: Buckling of bars at 3rd layer
- B2: Buckling of bars at 2nd layer
- B1: Buckling of bars at 1st layer

**Graph Descriptions:**
- **A-2H-f4p3ld5-NR25**
- **A-2H-CR25-f4p3ld5-NR25**
- **A-2H-CR50-f4p3ld5-NR25**

**Graph Details:**
- Lateral Capacity, V (kips) vs. Displacement, Δ (in)
- Symbols and markers indicate different layers and their corresponding states.
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<td>4.141 80.11  Max</td>
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UC: Un-corroded specimen

A: All-sides corroded specimen

H: Corrosion height

Ld: Shear span to depth ratio

NR: Axial load ratio

YT: Yielding of transverse bars

Y1: Yielding of bars at 1st layer

CR: Area loss

C: Crushing of Concrete

Max: Maximum lateral capacity

B: Buckling of bars

B4: Buckling of bars at 4th layer

B3: Buckling of bars at 3rd layer

S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
CR: Area loss
CR: Area loss
La: Shear span to depth ratio
NR: Axial load ratio
Max: Maximum lateral capacity
B4: Buckling of bars at 4th layer
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
CR: Area loss  
S1: Straightening of buckled bars at 1st layer  
NR: Axial load ratio  
Ld: Shear span to depth ratio  
ρ: Steel reinforcing ratio  
f: Compressive strength of concrete  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
S1: Straightening of buckled bars at 1st layer  
2Ld: Shear span to depth ratio  
ρ: Steel reinforcing ratio  
f: Compressive strength of concrete
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UC: Un-corroded specimen  
A: All-sides corroded specimen  
B: Buckling of bars at 4th layer  
S: Straightening of buckled bars at 1st layer  
YT: Yielding of transverse bars  
C: Crushing of Concrete  
Max: Maximum lateral capacity  

UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
Y1: Yielding of bars at 1st layer  
B3: Buckling of bars at 3rd layer  
S1: Straightening of buckled bars at 1st layer  
Y1: Yielding of bars at 1st layer  
CR50: Area loss  
Y1: Yielding of bars at 1st layer  
CR25: Area loss  
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CR: Area loss
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
Ld: Shear span to depth ratio
NR: Axial load ratio
F: Compressive strength of concrete
ρ: Steel reinforcing ratio
Y: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
Y2: Yielding of bars at 2nd layer
Y3: Yielding of bars at 3rd layer
Y4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
B4: Buckling of bars at 4th layer
C: Crushing of Concrete
Max: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer

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UC: Un-corroded specimen  
A: All-sides corroded specimen  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of concrete  
Max: Maximum lateral capacity  
CR: Area loss  
L: Shear span to depth ratio  
NR: Axial load ratio
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UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
F: Compressive strength of concrete  
p: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
F: Compressive strength of concrete
p: Steel reinforcing ratio
 Δ: Yielding of bars at 1st layer
 B: Yielding of bars at 3rd layer
 H: Corrosion height
 Ld: Shear span to depth ratio
 NR: Axial load ratio
 CR: Area loss ratio
 Max: Maximum lateral capacity

B1: Buckling of bars at 4th layer
B2: Buckling of bars at 3rd layer
B3: Buckling of bars at 1st layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
Y1: Yielding of bars at 1st layer
V: Yielding of bars at 4th layer
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
C: Crushing of Concrete
Max: Maximum lateral capacity
NR: Axial load ratio

Lateral Capacity, V (kips) vs. Displacement, Δ (in)
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UC: Un-corroded specimen  
A: All-sides corroded specimen  
P: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
υ1: Yielding of bars at 1st layer  
υ3: Yielding of bars at 3rd layer  
υ4: Yielding of bars at 4th layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
Lateral Capacity, V (kip)  
Displacement, Δ (in)  

UC-f7p3Ld5-NR05  
A-1H-CR25-f7p3Ld5-NR05  
A-1H-CR50-f7p3Ld5-NR05  

UC: Un-corroded specimen  
A: All-sides corroded specimen  
P: Compressive strength of concrete  
ρ: Steel reinforcing ratio  
υ1: Yielding of bars at 1st layer  
υ3: Yielding of bars at 3rd layer  
υ4: Yielding of bars at 4th layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
Lateral Capacity, V (kip)  
Displacement, Δ (in)
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
F: Compressive strength of concrete
p: Steel reinforcing ratio
B: Buckling of bars
Ld: Shear span to depth ratio
NR: Axial load ratio
Y: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
Max: Maximum lateral capacity
C: Crushing of Concrete
S1: Straightening of buckled bars at 1st layer
Max: Maximum lateral capacity
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
f: Compressive strength of concrete
p: Steel reinforcing ratio
l: Shear span to depth ratio
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
F: Compressive strength of concrete
p: Steel reinforcing ratio
Y1: Yielding of bars at 1st layer
Y: Yielding of transverse bars
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
C: Crushing of Concrete
Max: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer
S2: Straightening of buckled bars at 2nd layer
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<th>Layer</th>
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<th>Lateral Capacity, ( V ) (kips)</th>
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

YT: Yielding of transverse bars
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
S1: Straightening of buckled bars at 1st layer

ρ: Steel reinforcing ratio
Ld: Shear span to depth ratio
NR: Axial load ratio

\( f \): Compressive strength of concrete

\( \rho \): Steel reinforcing ratio

\( \Delta \): Displacement

\( V \): Lateral capacity

\( \Delta \): Displacement

\( V \): Lateral capacity
<table>
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<td>132.82 Max</td>
<td>1.663 54.87 Max</td>
<td>1.522 46.18 Max</td>
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</table>

UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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**UC:** Un-corroded specimen  
**A:** All-sides corroded specimen  
**F:** Compressive strength of concrete  
**p:** Steel reinforcing ratio  
**YT:** Yielding of transverse bars  
**S:** Yielding of bars at 1st layer  
**T:** Crushing of Concrete  
**Y1:** Yielding of bars at 1st layer  
**C:** Crushing of Concrete  
**S1:** Straightening of buckled bars at 1st layer  
**YTC:** Yielding of transverse bars at 1st layer  
**CR:** Area loss  
**Ld:** Shear span to depth ratio  
**NR:** Axial load ratio  
**Max:** Maximum lateral capacity  

![Graph A-2H-f7p3Ld5-NR25](image-url)
<table>
<thead>
<tr>
<th>Layer</th>
<th>UC-f4p3Ld2.5-NR00</th>
<th>A-1H-CR25-f4p3Ld2.5-NR00</th>
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UC: Un-corroded specimen
F: Compressive strength of concrete
Y1: Yielding of bars at 1st layer
B4: Buckling of bars at 4th layer
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B3: Buckling of bars at 3rd layer
S1: Straightening of buckled bars at 1st layer

Displacement, $\Delta$ (in)
UC-f4p3Ld2.5-NR05 | A-1H-CR25-f4p3Ld2.5-NR05 | A-1H-CR50-f4p3Ld2.5-NR05

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<td>B4</td>
<td>0.110</td>
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
LD: Shear span to depth ratio
NR: Axial load ratio
Max: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer
<table>
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<td>Δ  V Status</td>
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<td>186.25 C</td>
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<td>0.767</td>
<td>193.66 Max</td>
<td>0.597 83.42 Max</td>
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UC: Un-corroded specimen  
A: All-sides corroded specimen  
P: Compressive strength of concrete  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
Max: Maximum lateral capacity  
R: Area loss  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
S1: Straightening of buckled bars at 1st layer
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<tr>
<td>4</td>
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<td>0.399 106.84 Max</td>
<td>0.466 87.47 Max</td>
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
CR: Area loss
Ld: Shear span to depth ratio
NR: Axial load ratio
Max: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer

Y1: Yielding of bars at 1st layer
B4: Buckling of bars at 4th layer
C: Crushing of Concrete
B3: Buckling of bars at 3rd layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
f: Compressive strength of concrete
p: Steel reinforcing ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
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B3: Buckling of bars at 3rd layer
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UC-4p3ld2.5-NR00
A-2H-CR25-4p3ld2.5-NR00
A-2H-CR50-4p3ld2.5-NR00

Displacement, Δ (in) vs. Lateral Capacity, V (kip)

UC: Un-corroded specimen
A: All-sides corroded specimen
f: Compressive strength of concrete
p: Steel reinforcing ratio
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
S1: Straightening of buckled bars at 1st layer

Displacement, Δ (in) vs. Lateral Capacity, V (kip)
<table>
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<th>V</th>
<th>Status</th>
<th>Δ</th>
<th>V</th>
<th>Status</th>
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<td>0.296</td>
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<td>0.651</td>
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<td>177.36</td>
<td>Max</td>
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<td>97.30</td>
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UC: Un-corroded specimen
A: All-sides corroded specimen
f: Compressive strength of concrete
Y1: Yielding of bars at 1st layer
B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
C: Crushing of Concrete
Max: Maximum lateral capacity
NR: Axial load ratio
S1: Straightening of buckled bars at 1st layer
<table>
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A-2H-f4p3Ld2.5-NR15

- UC: Un-corroded specimen
- A: All-sides corroded specimen
- \( f \): Compressive strength of concrete
- \( \rho \): Steel reinforcing ratio
- \( Ld \): Shear span to depth ratio
- \( NR \): Axial load ratio
- \( CR \): Area loss
- \( Y1 \): Yielding of bars at 1st layer
- \( C \): Crushing of Concrete
- \( Max \): Maximum lateral capacity
- \( B4 \): Buckling of bars at 4th layer
- \( B3 \): Buckling of bars at 3rd layer
- \( B2 \): Buckling of bars at 2nd layer
- \( S1 \): Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen  
F: Compressive strength of concrete  
Y1: Yielding of bars at 1st layer  
B4: Buckling of bars at 4th layer  
A: All-sides corroded specimen  
P: Steel reinforcing ratio  
C: Crushing of Concrete  
H: Corrosion height  
Ld: Shear span to depth ratio  
CR: Area loss  
NR: Axial load ratio  
S1: Maximum lateral capacity  
Max: Maximum lateral capacity  
UC: Un-corroded specimen  
CR: Area loss  
NR: Axial load ratio  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
F: Compressive strength of concrete
Y1: Yielding of bars at 1st layer
B4: Buckling of bars at 4th layer
C: Crushing of Concrete
Max: Maximum lateral capacity

Displacement, Δ (in)

Lateral Capacity, V (kips)
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<td>Δ V</td>
<td>Status</td>
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<td>Y1 0.266</td>
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<td>0.671</td>
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<td>0.828</td>
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</table>

UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
D: Shear span to depth ratio  
NR: Axial load ratio  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
Y1: Yielding of bars at 1st layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
<table>
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<tr>
<th></th>
<th>UC-f4p4Ld2.5-NR15</th>
<th>A-1H-CR25-f4p4Ld2.5-NR15</th>
<th>A-1H-CR50-f4p4Ld2.5-NR15</th>
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<tr>
<td>Δ V</td>
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<td>Status</td>
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<tr>
<td>0.212</td>
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<td>Y1</td>
<td>0.385</td>
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<td>C</td>
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<td>0.688</td>
<td>207.96</td>
<td>Max</td>
<td>0.649</td>
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</table>

UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
C: Crushing of Concrete  
B: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
B4: Buckling of bars at 4th layer  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
Ld: Shear span to depth ratio  
CR25: Area loss  
NR: Axial load ratio  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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<td>Δ V Status</td>
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<td>0.155 77.33 B4</td>
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<td>0.381 120.30 B3</td>
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<td>0.537 135.55 YT</td>
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<td>0.554 221.78 Y1</td>
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<tr>
<td>0.536 220.08 C</td>
<td>0.430 97.96 C</td>
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<tr>
<td>0.608 224.50 Max</td>
<td>0.667 138.94 Max</td>
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UC: Un-corroded specimen  
f: Compressive strength of concrete  
p: Steel reinforcing ratio  
H: Corrosion height  
Ld: Shear span to depth ratio  
NR: Axial load ratio  
YT: Yielding of transverse bars  
Y1: Yielding of bars at 1st layer  
B4: Buckling of bars at 4th layer  
B3: Buckling of bars at 3rd layer  
C: Crushing of Concrete  
Max: Maximum lateral capacity  
S1: Straightening of buckled bars at 1st layer
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UC: Un-corroded specimen
A: All-sides corroded specimen
H: Corrosion height
Ld: Shear span to depth ratio
CR: Area loss
NR: Axial load ratio
YT: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
C: Crushing of Concrete
Max: Maximum lateral capacity

B4: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
S1: Straightening of buckled bars at 1st layer
<table>
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<th>UC-f4p4Ld2.5-NR05</th>
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<td>Δ V</td>
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<td>Δ V</td>
<td>Status</td>
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<tr>
<td>0.181</td>
<td>45.30</td>
<td>B4</td>
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<td>0.514</td>
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<td>0.788</td>
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<td>YT</td>
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<td>191.13</td>
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<tr>
<td>0.828</td>
<td>194.27</td>
<td>Max</td>
<td>0.893</td>
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</table>

A-2H-f4p4Ld2.5-NR05

UC: Un-corroded specimen
A: All-sides corroded specimen
F: Compressive strength of concrete
p: Steel reinforcing ratio
Y: Yielding of transverse bars
Y1: Yielding of bars at 1st layer
B: Buckling of bars at 4th layer
B3: Buckling of bars at 3rd layer
B4: Buckling of bars at 4th layer
H: Corrosion height
Ld: Shear span to depth ratio
C: Crushing of concrete
NR: Axial load ratio
Max: Maximum lateral capacity
S1: Straightening of buckled bars at 1st layer
<table>
<thead>
<tr>
<th>Layer</th>
<th>UC-f4p4Ld2.5-NR15</th>
<th>A-2H-CR25-f4p4Ld2.5-NR15</th>
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<tr>
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<td>0.188</td>
<td>52.49 B3</td>
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<tr>
<td>0.679</td>
<td>100.66 YT</td>
<td>0.712 83.861 YT</td>
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<tr>
<td>0.527</td>
<td>199.61 Y1</td>
<td>0.624 80.931 Y1</td>
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</tr>
<tr>
<td>0.655</td>
<td>207.10 C</td>
<td>0.565 77.62 C</td>
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<tr>
<td>0.688</td>
<td>207.96 Max</td>
<td>0.839 104.60 Max</td>
<td>0.712 83.86 Max</td>
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</table>

UC: Un-corroded specimen  
A: All-sides corroded specimen  
H: Corrosion height  
CR: Area loss  
F: Compressive strength of concrete  
Y1: Yielding of transverse bars  
B: Yielding of bars at 1st layer  
B3: Buckling of bars at 3rd layer  
B4: Buckling of bars at 4th layer  
P: Steel reinforcing ratio  
Ld: Shear span to depth ratio  
C: Crushing of Concrete  
NR: Axial load ratio  
S1: Straightening of buckled bars at 1st layer
### Table 1: Lateral Capacity and Displacement Data

<table>
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<th></th>
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<tr>
<td><strong>Δ V Status</strong></td>
<td><strong>Δ V Status</strong></td>
<td><strong>Δ V Status</strong></td>
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<tr>
<td>0.030 13.97 B4</td>
<td>0.000 0.00 All Buckled</td>
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<tr>
<td>0.060 23.37 B3</td>
<td>0.307 55.91 S1</td>
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<tr>
<td>0.385 75.46 B2</td>
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<td>0.452 67.67 YT</td>
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<tr>
<td>0.464 81.65 YT</td>
<td></td>
<td>0.554 221.78 Y1</td>
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<td>0.536 220.08 C</td>
<td>0.484 82.83 C</td>
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<tr>
<td>0.608 224.50 Max</td>
<td>0.623 87.38 Max</td>
<td>0.554 221.78 Y1</td>
</tr>
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</table>

**Legends:**
- UC: Un-corroded specimen
- A: All-sides corroded specimen
- H: Corrosion height
- L: Shear span to depth ratio
- CR: Area loss
- NR: Axial load ratio
- C: Crushing of Concrete
- Y1: Yielding of transverse bars
- YT: Yielding of bars at 1st layer
- B4: Buckling of bars at 4th layer
- B3: Buckling of bars at 3rd layer
- S1: Straightening of buckled bars at 1st layer
- Max: Maximum lateral capacity

### Diagram 1: Lateral Capacity vs. Displacement for A-2H-f4p4Ld2.5-NR25

- UC-f4p4Ld2.5-NR25
- A-2H-CR25-f4p4Ld2.5-NR25
- A-2H-CR50-f4p4Ld2.5-NR25

**Legend:**
- UC-f4p4Ld2.5-NR25
- A-2H-CR25-f4p4Ld2.5-NR25
- A-2H-CR50-f4p4Ld2.5-NR25
- YT: Yielding of transverse bars
- C: Crushing of Concrete
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EDUCATION:

Ph.D., Structural Engineering
Aug. 2016
Syracuse University, Syracuse, NY
Full-time student with Teaching Assistantship

- Dissertation Title: “Effect of Severe Corrosion on Lateral Strength of Square RC Bridge Columns”

M.S., Structural Engineering
Jan. 2008
University of Tabriz, Tabriz, Iran

B.S., Civil Engineering  Sept. 2004

Azarbaijan University of Tarbiat Moallem, Tabriz, Iran


**TEACHING EXPERIENCE:**

**Teaching Assistant** at Syracuse University, Syracuse, NY


Holding recitation classes and labs, taking quizzes and grading:

- Design of Concrete Structures
- Statics
- Mechanics of Solids

**Lecturer** at Seraj Higher Education Institute and Azad University, Tabriz, Iran

Feb. 2008- Jul. 2010

Teaching courses at undergraduate level:

- Application of computation in Civil Eng. (OpenSees Navigator, SAP2000, ETABS2000 & SAFE), concrete technology, applied loads of structures, concrete and steel design of structures, landscaping, surveying, constructional machinery, drawing and map reading.

**RESEARCH ACTIVITIES:**

- Flexural Strength of Corroded Lapped Spliced RC Bridge Column Section
- Seismic evaluation and retrofit of deteriorated concrete bridge components
- Seismic Retrofit of Reinforced Concrete Columns with Steel Jackets
- Investigation on seismic behavior of columns in different conditional situations using "Opensees" and "Sap2000"
- Seismic Retrofit of masonry structures
PRESENTATIONS:


S. Sotoud, R.S. Aboutaha. "Flexural Strength of Corroded Lapped Spliced RC Bridge Column Section", ASCE Structures Congress, Boston, USA, 2014


S. Sotoud, B. Farahmand. "Seismic Retrofit of Reinforced Concrete Columns with Steel Jackets", 8th International Congress on Civil Engineering, Shiraz, Iran, 2009

S. Sotoud. "Seismic Retrofit of Reinforced Concrete Columns with Steel Jackets of Different Thicknesses", 1st International Conference on Concrete Technology, Tabriz, Iran, 2009

PROFESSIONAL EXPERIENCE:

“Pey Afkan Sazeh” Consulting Engineers Company, Tabriz, Iran

July. 2009-July. 2010

As a designer of concrete and steel buildings:

- Designed concrete and steel buildings with detailed drawings
- Designed “Laleh Motel”, a concrete structure with detailed drawings
- Designed a storage truss and an office building and the connection between them
As senior expert of retrofit section:

- Analyzed the seismic behavior of school buildings
- Suggested three retrofit methods such as shotcreting the walls, adding concrete shear walls or using FRP to strengthen the walls for masonry structures and adding bracing system, adding shear walls or strengthening individual elements for steel structures
- Estimated cost of proposed retrofit designs
- Selected the optimized method
- Supervision of construction

As a technical engineer:

- Supervised “Be’sat Apartments”, concrete buildings
- Checked measurement and evaluation of contractor

As a designer of concrete and steel buildings:

- Designed “Betaja Apartments”, concrete buildings with detailed drawings

**Honors and Activities:**

- Recipient of “Syracuse University Outstanding Teaching Assistant Award”, 2016
- Certified “Engineering in Training” exam, 2015
- Won “Wen-Hsiung and Kuan-Ming Li” Graduate Fellowship, awarded 2014
- Member of “Phi Beta Delta” International honor society, awarded 2014
Certified in “Women in Science and Technology (WiSE)” program, awarded 2014

Leader of Graduate Students Orientation, College of Engineering and Computer Science, 2012

Member of ASCE, ACI

First place of "Egg Protection Device" in ACI concrete contests, Iran ACI chapter, Tehran, 2005

6th place of "Concrete Cube Competition" in ACI concrete contests, Iran ACI chapter, Tehran, 2003

**SKILLS:**

**Software:** Sap2000, Etabs2000, Safe, STAAD.Pro
AutoCAD, Revit
Microsoft Word, Microsoft Excel, Microsoft Power Point
Opensees
ANSYS, ABAQUS
MathCAD

**Language:**

English Fluent
Persian (Farsi) Native
Azarbaijani Native
Turkish Fluent
Arabic Basic

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Laura J. and L. Douglas Meredith Professor for Teaching Excellence
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