

Computational modeling of the axial behavior of corroded and buckled short steel piles strengthened using concrete-filled GFRP jackets

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ABSTRACT

Steel columns or piles commonly used in bridges and marine structures in various parts of the world exhibit insufficient strength due to increased load demands or section loss due to corrosion. The latter issue can cause unexpected buckling of these columns. In this study, the repair of corroded and buckled short steel piles using concrete-filled GFRP jackets was numerically simulated to better understand the overall effectiveness of the method and the influence of the parameters involved. Thirteen steel piles used in this study had different degradation characteristics to investigate the section losses affecting the axial capacity and failure modes. The parameters considered included the presence or absence of concrete steel reinforcement (rebar), the diameter of the rebar, the number of GFRP layers, the location of the worn zone in the steel section and the severity of the section loss. The numerical models were validated using these experimental results of the repaired specimens from the authors' earlier work on the topic. The main contribution of this study is in the development of the numerical models that can very accurately simulate the ultimate strength and damage behavior of retrofitted piles that are heavily corroded and buckled. The validated models were used in a parametric study to understand the effects of different levels of concrete strength, or corrosion of the piles in the flanges, the web, or both. The results of the parametric study provided further understanding of the structural behavior of degraded and strengthened steel piles.

1. Introduction

Steel H-piles are widely used in bridges, marine structures, and industrial facilities. Due to their continuous exposure to chloride and moisture, these steel members are susceptible to corrosion. The deterioration due to corrosion can be severe but generally localized within the section at the water or soil level. Field observations indicate that the corrosion causes severe reduction in the flange and web thicknesses of H-piles; and in some cases, a near-total loss of the cross section such as voids in the web and flanges are observed. Besides, these corroded steel piles may suffer premature global or local buckling due to a decrease in their capacity [1]. Since the corrosion of these critical load-bearing members could cause partial or complete structural collapse, an urgent repair and strengthening may be required when a deficiency is noticed [2–6]. To effectively repair these steel piles, it is necessary to accurately predict the

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load-bearing capacity before and after repair depending on the type and extent of damage.

Fiber reinforced polymers (FRP) have increasingly been preferred as an alternative to traditional repair materials due to their advantages such as low cost, reduced time and labor associated with the application, low strength-to-weight ratio and high corrosion resistance. Various repair and strengthening techniques are available with FRP. One of the most common and efficient repair methods is the concrete/grout-filled FRP jackets for corroded and buckled steel piles. In this system, the FRP jacket provides confinement to the concrete core used to stabilize the steel piles. Additionally, it prevents or delays the premature buckling of steel piles and increases the bearing capacity. This strengthening method may be applied throughout the entire height of a pile as well as locally at the corroded section or sections.

Recently, several experimental [2,6–14], numerical [4,15–17] and analytical [8,18,19] studies have been conducted on concrete-filled FRP jackets. Liu et al. [8] experimentally and analytically evaluated the efficiency and feasibility of the FRP-jacket strengthening. Seven 3 m long S4x9.5 steel members were tested under compression, two of which were control. The flanges of the members were notched to represent the loss of cross section due to corrosion and then repaired with glass FRP (GFRP) pipes that were filled with concrete. The repaired members were loaded axially, and global buckling occurred just outside the repair region. It was found that the ultimate bearing capacity of the retrofitted members was as much as twice that of the undamaged reference specimens. It was also noted that the use of lightweight and expansive concrete as a core material had a positive effect on the capacity. An analytical model was also proposed in the same study to estimate the load-bearing capacity of steel piles using the FRP-jacket repair technique. El-Tawil and Ekiz [11] proposed a new technique to delay or prevent the buckling of single and double angle steel braces. The pre-fabricated mortar blocks were placed inside the legs of the steel angles to form a cementitious core. Then, the recommended strengthening method was completed by wrapping the entire system with carbon FRP (CFRP). Seven CFRP strengthened samples were subjected to reversed axial loading in a specially designed frame and the energy dissipation was investigated. The strengthening system prevented the buckling of the braces up to 2% inter-story drift level. The strengthening method nearly doubled the cumulative energy dissipation of the braces. In addition, it was observed that the strengthening method was more effective for double angle braces than single angle braces that are prone to premature out-of-plane buckling.

He et al. [20] tested a new type of hollow concrete-filled GFRP tube composite pile and the results were compared with finite element analysis (FEA). The effects of the hollow ratio, the diameter-thickness ratio of the GFRP jacket and the concrete strength were investigated through axial compression tests performed on 27 samples. These three parameters were analyzed using validated finite element models (FEM). An ultimate bearing capacity adjustment coefficient considering the effect of hollow sections was proposed. A formula presented in the article could accurately predict the ultimate bearing capacity of concrete-filled GFRP hollow tubular composite columns. Han et al. [21] studied the behavior of square and circular tubular piles strengthened with FRP under constant axial load and cyclically increasing bending load. The recommended strengthening method included a circular steel inner tube, a square or circular concrete core and a bi-directional CFRP outer tube. The failure in the piles occurred due to the rupture of the fibers. The number of FRP layers and the axial load level parameters were taken into consideration. The research findings indicated that the elastic-plastic stiffness of the specimen increased with the increasing number of layers, while the number of CFRP layers only slightly affected the elastic stiffness. The difference in the ultimate strength of square samples with one and two layers of CFRP was found as 12.2% while the same was 20.2% for circular samples. As the number of FRP layers increased, the strength of the piles increased while the flexural ductility decreased. In another study [22], 600 mm long, 91.5 mm × 91.5 mm square hollow concrete filled steel tubes were retrofitted with CFRP strips at varying spacing and they were subjected to axial compression. An analytical model was proposed to predict the axial load-bearing capacity of these concrete filled steel members. The results indicated that a reduction of FRP strip spacing can delay the local buckling of the steel members and increase their ultimate bearing capacity. There have been numerous studies investigating the stress-strain behavior of concrete confined with FRP under axial pressure. Ozbakkaloglu et al. [23] presented a comprehensive review of 88 models developed to predict the axial stress-strain behavior of FRP-confined concrete in circular sections. These studies were classified into two broad categories: design-oriented and analysis-oriented models. Realfonzo et al. [24] investigated the ultimate strain of the concrete confined with FRP. One of the main failure modes of the FRP-confined concrete system was found to be the stripping of the epoxy adhesive interface from the fiber layers hence affecting the bearing capacity [25]. Ultra-high-performance-concrete (UHPC) confined by FRP was also studied under axial compression loading for different eccentricities to investigate the compressive behavior [26]. The specimens generally showed a strain-hardening behavior. A gradual decrease in the load bearing capacity and deformability were observed as the loading eccentricity increased. It was found that the bearing and deformation capacity of the piles were increased due to the constraint caused by the FRP-confined concrete. In addition, the concrete-filled FRP jacket proposed as a repair system delayed or prevented the buckling of the steel piles. Based on the regression analysis of the experimental and numerical results, a design equation was proposed to determine the ultimate load bearing capacity of UHPC confined with FRP under eccentric loading.

The two main contribution of this paper to existing body of knowledge are (1) modeling of damaged structural elements, specifically, steel piles, which has been lacking in literature due to difficulties associated with representing the deformed geometry, and initial conditions of damaged structural components, and (2) a lack of understanding of the structural behavior of degraded and strengthened structural members. These two knowledge gaps are addressed in this paper through (1) a validated numerical modeling approach, and (2) a parametric study using the validated models that has revealed the effectiveness of the repair approach for different levels of degradation, which would be very difficult and costly to study experimentally.

2. Research significance

Previous research has focused on the behavior of undamaged and damaged steel piles retrofitted with FRP. In practical

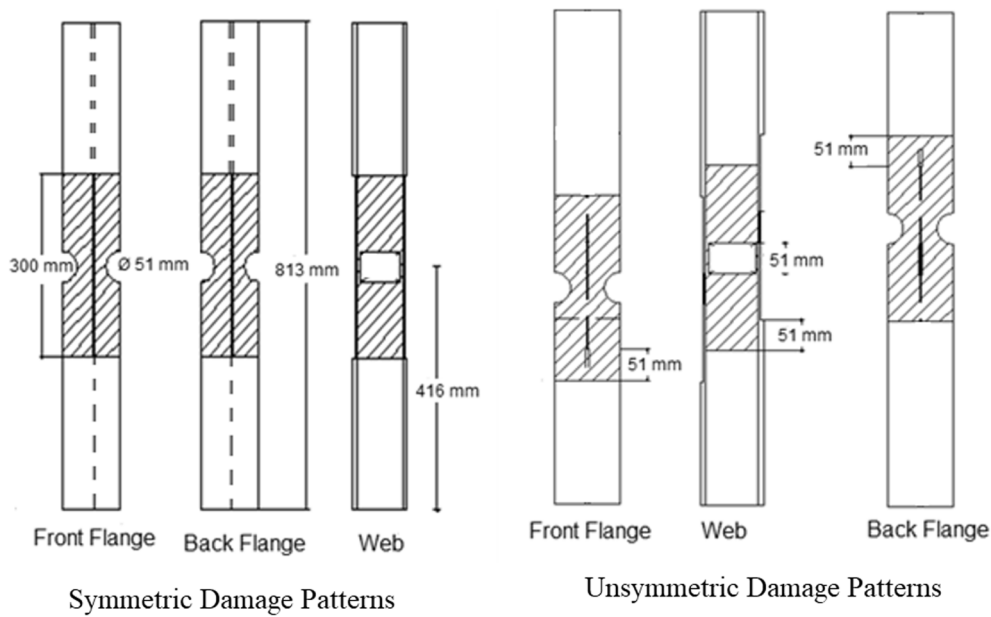


Fig. 1. Schematic representation of corrosion patterns of piles (adepted from [2]).

Table 1
Test matrix.

	Designation		Loss of Cross Section (%)	Initial Out of Straightness (mm)	Residual Load of Unrepaired Piles (kN)	Retrofitting Scheme	No of GFRP Layers	Rebar	Compressive Strength (kN)	
	Karagah et al. [2]	Kaya et al. [6]							Experimental	
									Unrepaired	Repaired
Group #1	0/0	G1/2/NR-1	0.0	15.1	809	Grout + GFRP	2	–	956	898.0
	0/30	G1/2/NR-2	9.0	7.9	769	Grout + GFRP	2	–	894	907.0
	0/60	G1/2/NR-3	16.1	11.1	408	Grout + GFRP	2	–	792	943.0
Group #2	50/0	G2/3/NR-1	39.9	11.1	408	Grout + GFRP	3	–	520	956.4
	50/30	G2/3/NR-2*	43.6	22.4	374	Grout + GFRP	3	–	578	872.3
Group #3	75/0	G3/2/4#3-1	58.9	6.4	325	Grout + GFRP + Rebar	2	4#3	409	912.0
	75/60	G3/2/4#3-2	67.6	4.0	205	Grout + GFRP + Rebar	2	4#3	311	731.0
	75/60/NV/US	G3/3/4#3	75.4	11.1	176	Grout + GFRP + Rebar	3	4#3	253	992.0
	75/60/NV/US/WR	G3/2/4#4	74.7	2.5	306	Grout + GFRP + Rebar	2	4#4	311	934.0
	75/60/V/S	G4/2/NR	78.8	4.0	67	Grout + GFRP	2	–	178	664.0
Group #4	75/60/V/S/WR	G4/2/4#4	88.3	3.2	64	Grout + GFRP + Rebar	2	4#4	160	298.0
	75/60/V/US	G4/3/NR	78.8	6.4	62	Grout + GFRP	3	–	178	840.0
	75/60/V/US/WR	G4/3/4#4	77.4	12.7	65	Grout + GFRP + Rebar	3	4#4	173	943.0

* Total length = 686 mm; repaired length = 521 mm

applications, local and global buckling are present in steel piles with severe corrosion due to a decrease in the axial capacity. Several repair techniques are available to stabilize deteriorated steel piles. However, the stabilization of the buckled piles may present challenges influencing the effectiveness and ease of installation of the repair system. For example, locally or globally buckled piles may

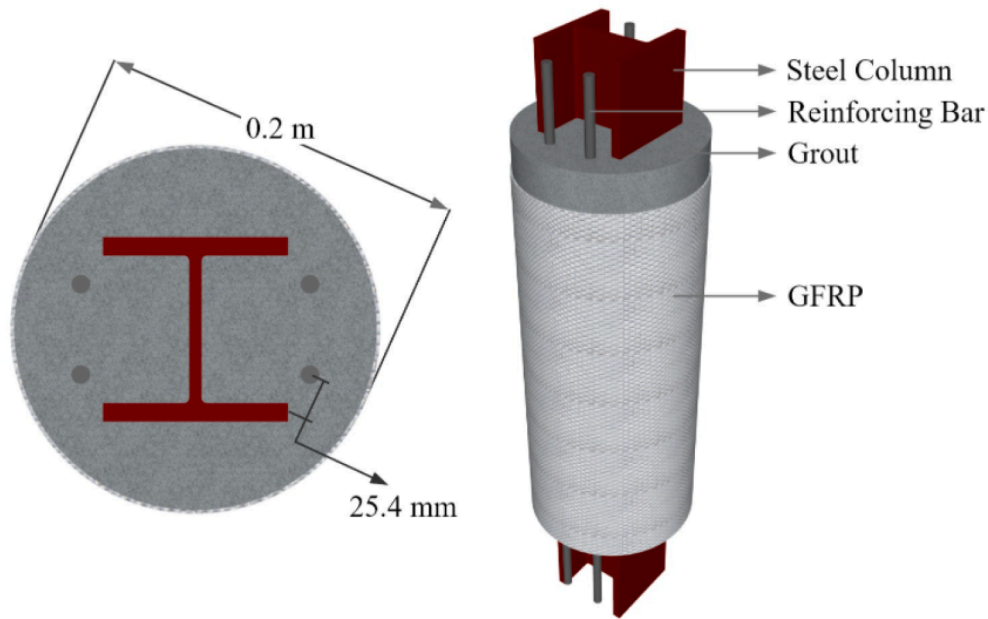


Fig. 2. Schematic representation of repaired steel piles.

exhibit larger residual transverse deflections and reduced residual strength than their unbuckled counterparts. This paper investigates the effectiveness of a GFRP-based system for rapid retrofit of buckled and corroded steel piles using numerical models. Non-linear FEAs were conducted to determine the remaining axial capacity of deteriorated steel piles as well as their post retrofit capacity. It is expected that the results of this paper will help develop comprehensive design guidelines for the retrofit of corroded and buckled piles. The common approach in literature is to use idealizations of the element geometry due to difficulties involved in realistic modeling of the damaged or deformed structural element. These assumptions reduce the accuracy of the computer models. As a new modeling approach, in this paper, first, a linear buckling analysis is performed to determine the Eigenvalues and elastic buckling modes. Next, a nonlinear buckling analysis is performed considering the defects and based on the obtained linear buckling modes as initial imperfections. In the third step, the geometric forms of the deformed steel piles obtained from the inelastic buckling analysis are integrated into the FEMs to make accurate predictions of the capacity of corroded and buckled short steel piles repaired with concrete-filled GFRP jackets.

3. Summary of experimental data

In this study, data from thirteen short steel piles that were tested by Karagah et al. [2] and Kaya et al. [6] were used. The steel piles were W4x13 (US designation). These piles with simulated local corrosion patterns were tested under axial compressive loading [2]. After testing, the buckled piles were retrofitted with concrete-filled GFRP jackets in various configurations and tested again to assess the effectiveness of the strengthening approach [6]. Configurations of the corrosion patterns that were tested are schematically shown in Fig. 1. In Karagah et al. [2], piles were assigned a five-part unique label. The first two sections of the label showed the percentage reduction in the flange and web thickness, respectively, while the third section indicated the presence or absence of a 51 mm void in the middle of the web to simulate the through web corrosion. The fourth part represented the symmetrical or unsymmetrical corrosion pattern as shown in Fig. 1. The last part showed the presence of a semi-circular cavity to simulate a flange void. The initial pile geometry reflects the deteriorated conditions of the member due to corrosion. Different damage patterns are shown in Fig. 1. The linear buckling analysis, which is the first step of the simulation, was conducted on these degraded members, and the simulation steps were continued from thereon as further explained in Section 4.

In the present study, according to the test results of Karagah et al. [2], the piles were divided into four groups considering the simulated corrosion patterns, damage modes, residual loads, and axial load–deflection responses. Table 1 summarizes the residual load, loss of cross section, retrofit method and initial out-of-straightness of the short steel piles used in this study for validation purposes. The residual load was determined based on previous test data. The initial out-of-straightness was calculated by measuring the maximum distance between the surface and the buckled piles after the piles were placed on a flat surface. The behavior of piles under axial load and data obtained from before retrofit testing are described in more detail in [2].

After testing the steel piles degraded by simulated local corrosion, the buckled piles were retrofitted with concrete-filled GFRP jackets in various configurations in [6]. The effect of two variables, the number of layers of the GFRP jacket and the supplemental longitudinal steel reinforcement (rebar) inside the concrete core, on the axial compressive properties of the buckled piles were investigated. Additional identifiers consisting of four parts were assigned to the repaired specimens (see Table 1). The first section of

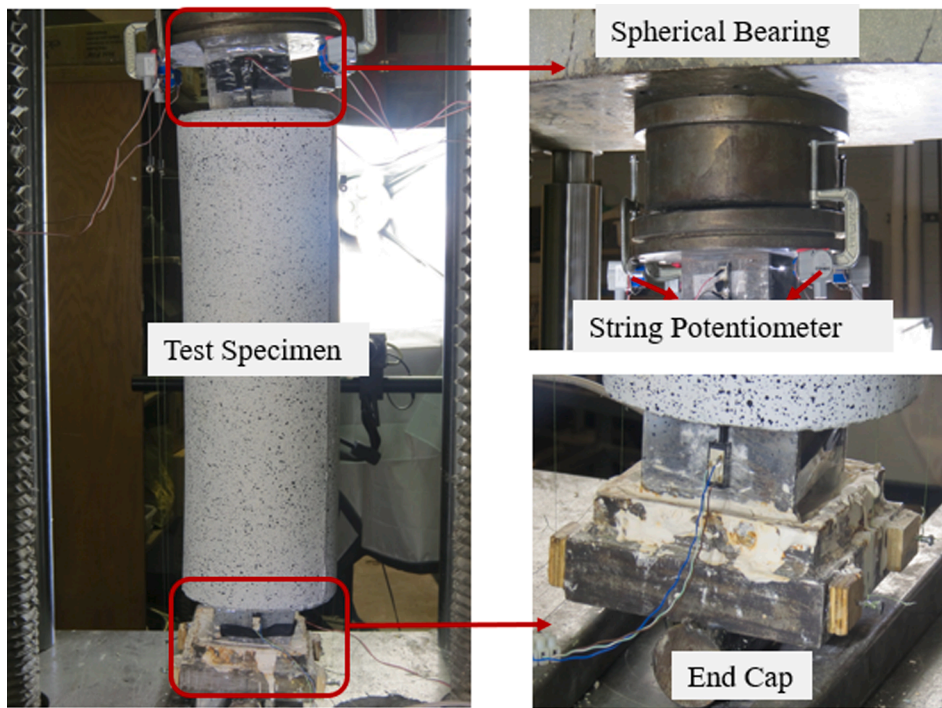


Fig. 3. Test setup.

the identifiers shows the specific group number of the specimens (G1, G2, G3 or G4). The second section of the identifier shows the number of GFRP layers in the jacket (two or three). The third part indicates whether there is internal rebar or not. If there is internal rebar, it shows the number and size of reinforcing bars (4#3 or 4#4). Reinforcing bar sizes are according to the US definitions. The nominal diameters of #3 and #4 reinforcing bars are 9.5 and 12.7 mm, as the US designation indicates the diameter of the bars in 1/8-inch (1 in. = 25.4 mm) increments. The fourth part is the serial number, which indicates multiple replicates of the same test specimen. Multiple iterations were performed to evaluate the reproducibility of the results due to uncertainties associated with the strengthening and testing of the buckled steel piles.

3.1. Test specimen design

The GFRP jacket applied in this study was wrapped around the pile as a single piece and in multiple layers. The flexible GFRP laminate was bonded to itself with epoxy to form a circular jacket. One of the important benefits of this proposed system is that the GFRP jacket can be easily applied on site in desired sizes. The repair consists of nine stages: (1) the buckled steel pile surface was cleaned with a wire brush and alcohol; (2) GFRP sheets were prepared by cutting to size taking into account a 203 mm of overlap; (3) wooden spacers with a diameter of 50 mm were placed on the flanges of the steel piles to prevent the GFRP jacket from contacting the steel pile and maintain a uniform thickness of the concrete; (4) the two-component epoxy was mixed with a low speed mixer for 3 min until a homogeneous consistency was achieved; (5) The prepared epoxy was applied to both sides of the GFRP sheet; (6) GFRP jackets were wrapped around the steel piles to a nominal diameter of 203 mm; (7) plastic zip ties were used for 48 h to keep the GFRP jacket stable until the epoxy cured; (8) rebar, if any, were placed and fixed inside the GFRP jacket; and (9) an expansive grout was prepared and poured into the GFRP jacket. A schematic representation of the repaired specimens is shown in Fig. 2.

3.2. Loading scheme and instrumentation

The specimens were tested using a 1780 kN Tinius-Olsen universal testing machine under monotonic compression load. The top and bottom of the specimens were designed to provide a nominally fixed/pinned condition about the strong axis and a nominally simply supported condition about the weak axis. Gaps were formed between the specimen and the support plates because damaged specimens were used. A thin layer of self-leveling plaster was applied to the loading points to avoid an uneven application of forces at the ends of the specimens. Before loading, the pile was placed in the center of the loading device to ensure that the pile is under concentric axial compression as shown in Fig. 3. All specimens were subjected to the same loading protocol. Up to 1330 N of loading, a displacement ramp rate of 5.1 mm/min was used. The actual loading of the specimens was assumed to start at 1330 N, while the earlier stages were considered as the seating of the specimens. The force was held constant for two minutes to check the system at the preload level. In the second phase of loading, the ramp speed was reduced to 0.2 mm/min. The testing was continued by increasing the load

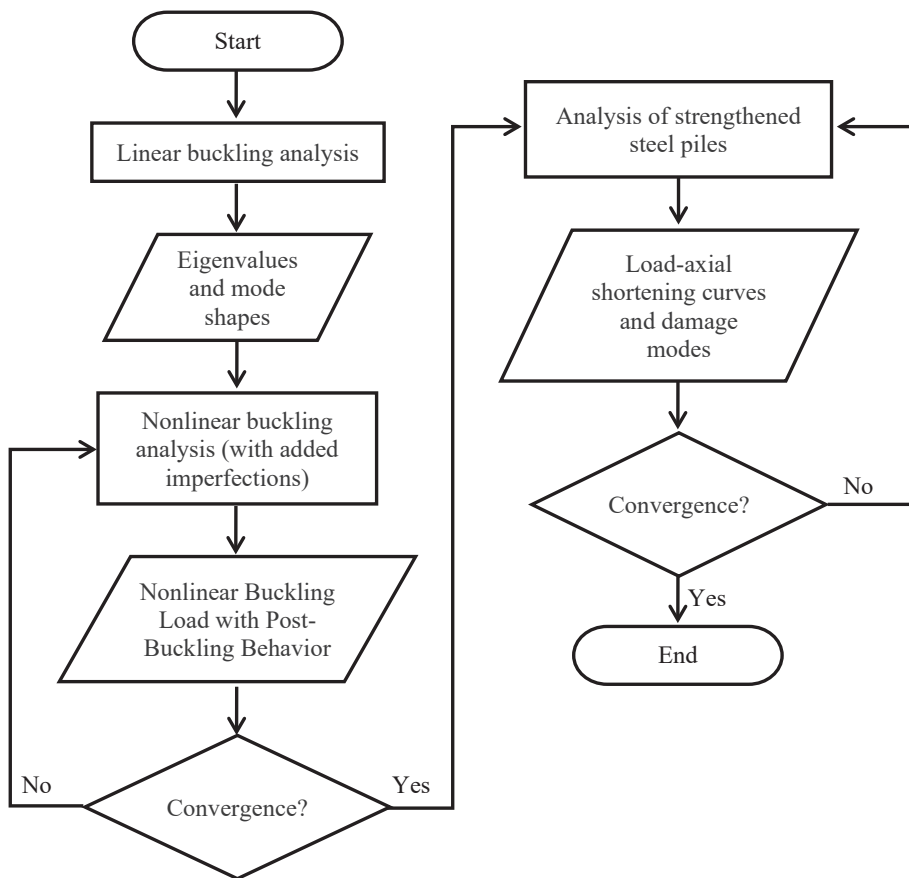


Fig. 4. Flowchart of modeling in ABAQUS.

monotonically until the specimen failed or excessive lateral deformations were observed. Four string potentiometers were placed at the four corners to measure the axial shortening between the two end caps. Data were collected using a Micro-Measurements System 7000 data acquisition system.

4. Numerical study

The finite element analysis was performed in ABAQUS [27]. The concrete damaged plasticity (CDP) material model was used for the concrete, and the classical metal plasticity (CMP) model was used for the rebar and the structural steel sections. The Hashin model was used for the GFRP. Detailed information on the material models and modeling processes of the strengthened piles is given in the following sections.

4.1. Numerical modeling process

The numerical studies here are comprised of three main steps as shown in Fig. 4. In the first step, a linear buckling analysis was performed to determine the Eigenvalues and elastic buckling modes. In the second step, a nonlinear buckling analysis, which considers the imperfections defined based on the linear buckling modes, was performed. The initial imperfections were obtained from the linear buckling analysis and defined as the buckling mode shapes. The first critical global buckling mode was used to represent the global deformations, while the first local buckling mode was used to represent the local deformations. In the third step, the damage modes, obtained in the second step, were verified based on the damage conditions obtained in the experiments. The deformed steel piles obtained from the nonlinear buckling analysis were embedded in the circular concrete model. The “embedded region” constraint option in ABAQUS was used for this purpose. Similarly, the rebar was embedded in the concrete, assuming a complete bond between concrete and the deformed pile and rebar. The C3D8R general-purpose linear brick elements, with reduced integration, were used to model the concrete. The B31 beam elements were used for modeling the rebar. S4R shell elements with four nodes and reduced integration was used for modeling the GFRP jacket and the steel piles. As shown in Fig. 5, the boundary conditions were defined to replicate the experiments, that is one end was fixed, and the other end was free to rotate about the y-axis. The top and bottom regions of the steel piles were coupled by the “rigid body” constraint option to the defined reference points (RPs), and the boundary conditions

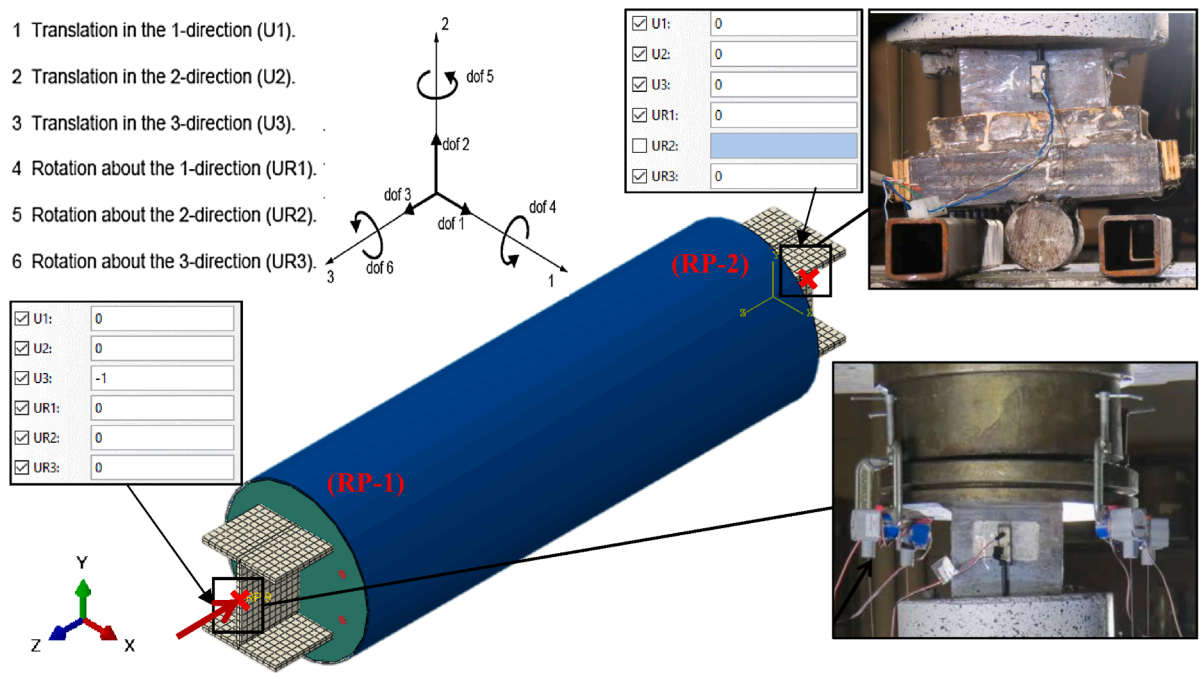


Fig. 5. Loading and boundary conditions used in numerical models.

Table 2

Mesh information for FE models.

Part Number	Finite Element Type	Number of Elements	Schematic Representation of Finite Element
1	S4R (shell elements with four nodes and reduced integration)	3709	
2	C3D8R (general-purpose linear brick elements, with reduced integration)	55,221	
3	B31 (two-node beam element)	316	
4	S4R (shell elements with four nodes and reduced integration)	6320	

were assigned to the RPs. In the z-axis direction, the loading was applied to RP-1. In Table 2, the finite element types used for the parts of the model that are shown in Fig. 6 are summarized with brief explanations. To determine the element size, a mesh convergence study was carried out for nonlinear buckling analysis of the steel piles. A suitable mesh size was determined as 8 mm for the steel pile that was a good compromise between computational efficiency and accuracy. The mesh sizes for the concrete, rebar, and GFRP were also taken as approximately 8 mm. Fig. 6 depicts various modeling phases in ABAQUS/CAE. Firstly, a linear buckling analysis was performed on the unbuckled columns machined to simulate different degrees and patterns of corrosion (Fig. 1) to determine the Eigenvalues and elastic buckling modes. The buckling modes obtained from the linear buckling analysis were used to determine the imperfections through scaling as per Eqn 1 below and used as inputs for the nonlinear buckling analysis.

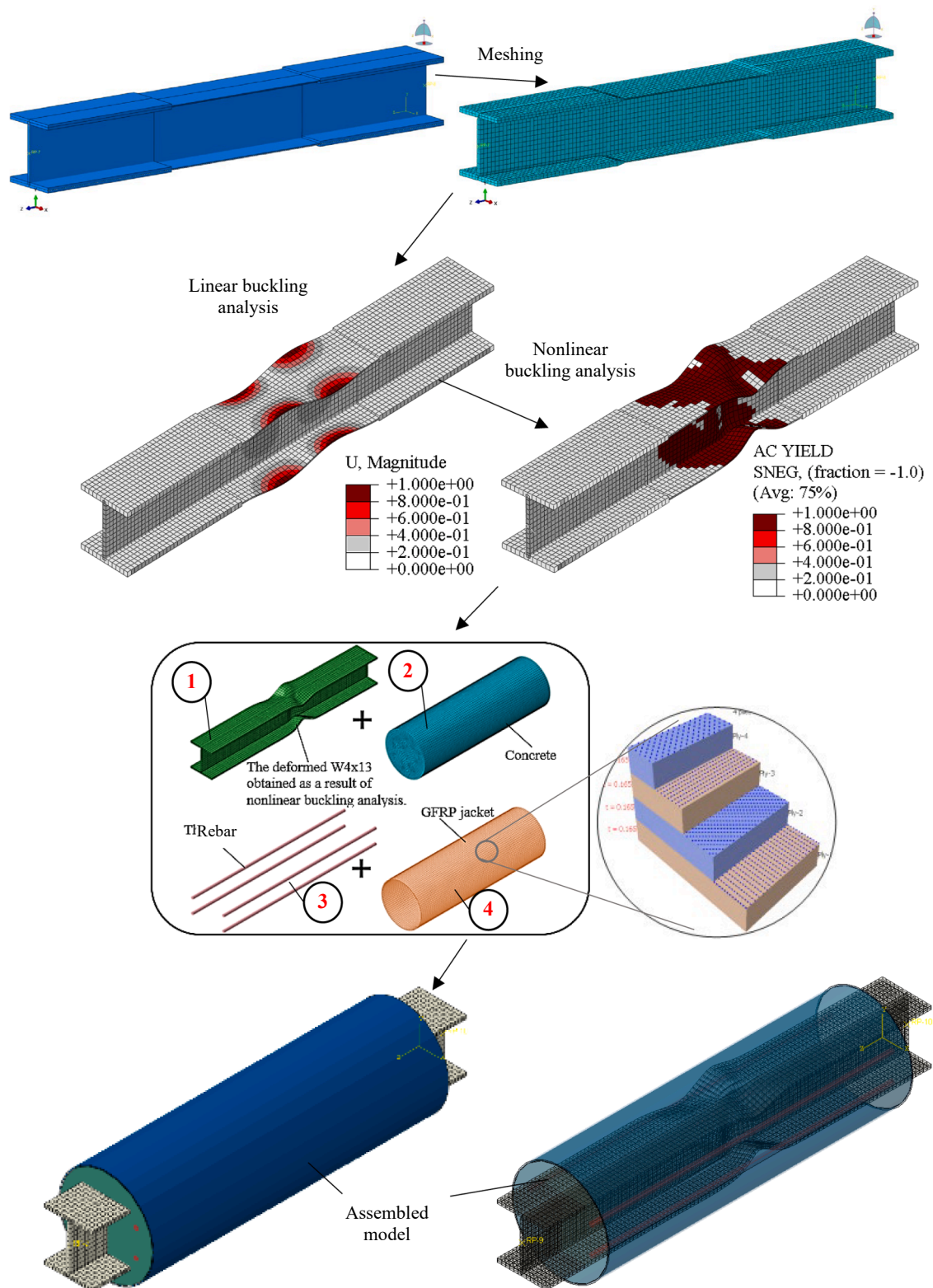


Fig. 6. Modeling steps in ABAQUS.

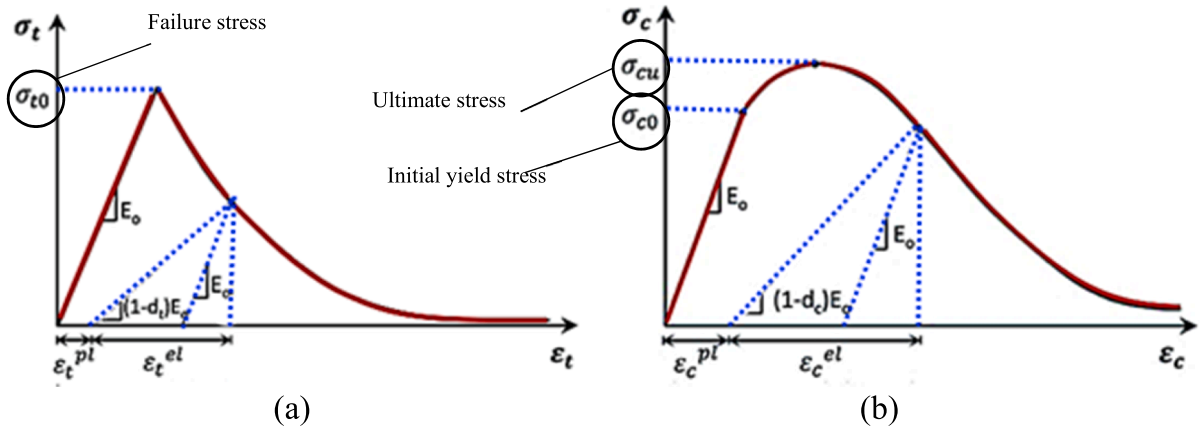


Fig. 7. Stress–strain diagrams of CDP model; (a), tension and (b), compression (reproduced from [27]).

Table 3

Proportions of concrete mixture and cylinder test results (from Kaya et al. [6]).

Casting Group	Mixture by Weight Ratio			Number of Cylinders Tested	Compressive Strength	
	Grout	Gravel	Water		Mean (MPa)	COV ^a
Group #1	1.0	0.5	0.17	9	51	4%
Group #2	1.0	0.5	0.15	5	63	3%
Group #3	1.0	0.5	0.15	5	71	3%
Group #4	1.0	0.5	0.15	4	59	2%

^a Coefficient of variation.

Table 4

Material properties of concrete for FEA.

Property	Group	Value
Modulus of elasticity, E_0 (MPa)	Group #1	37,000
	Group #2	39,000
	Group #3	41,000
	Group #4	39,000
	All groups	0.2
Poisson ratio, ν		2400
Density, ρ (kg/m ³)		36
Dilation angle, controls the amount of plastic volumetric strain created during plastic shearing, ψ		0.1
Eccentricity, e		0.99
Max. damage parameter of tension, d_t		0.99
Max. damage parameter of compression, d_c		0.6667
Ratio of the second stress invariant on the tensile meridian to that on the compressive meridian, K_c		1.16
Ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress, $\frac{f_{bo}}{f_{co}}$		0.0005
Viscosity parameter, μ		

$$\Delta_{si} = \sum_{i=1}^M \omega_i \cdot \varphi_i \quad (1)$$

In Eqn 1, φ_i is the i^{th} mode shape and ω_i is the associated scale factor [38]. The geometric forms of the deformed steel piles due to corrosion and buckling, which were derived from the inelastic buckling analysis were then incorporated into the models with different repair schemes using concrete-filled GFRP jackets in the final stage.

4.2. The material models

4.2.1. Concrete damaged plasticity (CDP)

Lubliner et al. [36] proposed the CDP model for static monotonic loading, which was later expanded by Lee and Fenves [37] to include dynamic and cyclic loading cases. The CDP material model is a continuum, plasticity-based, damage model for concrete. It assumes that the two main failure mechanisms are the tensile cracking and compressive crushing of concrete. The evolution of the yield

Table 5
Mechanical properties of steel piles (from Karagah et al. [2]).

Property	Flange	Web
Modulus of elasticity (GPa)	186	180
Yield strength (MPa)	387	439
Ultimate strength (MPa)	478	530
Strain at ultimate strength (mm/mm)	0.133	0.070

Table 6
Mechanical properties of reinforcing bars (from Kaya et al. [6]).

Property	Rebar #3Mean	Rebar #4Mean
Modulus of elasticity (GPa)	198	204
Yield strength (MPa)	437	399
Ultimate strength (MPa)	607	668

Table 7
Mechanical material properties of GFRP laminae for FEA (from Kaya et al. [6]).

Property	Longitudinal (0°)	Transverse (90°)
Tensile strength (MPa)	397	392
Tensile modulus (MPa)	21,918	18,416
Ultimate strain (%)	1.89	2.2
Poisson's ratio	0.28	0.26 ^a
Shear modulus (MPa)	1731	

^a Calculated as $\nu_{ij}=(E_i*\nu_{ji})/E_j$ based on symmetry of the material stiffness matrix [34].

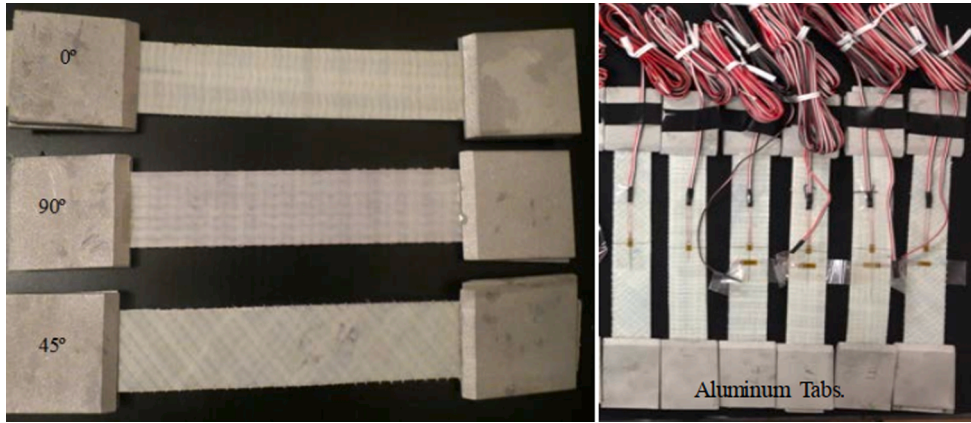


Fig. 8. GFRP coupons.

Table 8
Properties of the adhesive (Quakewrap, N.D.).

Property	Value
Tensile strength (MPa)	30
Tensile modulus (MPa)	2268
Compressive strength (MPa)	55
Compressive modulus (MPa)	1923
Flexural strength (MPa)	55
Flexural modulus (MPa)	1725
Shear strength (MPa)	10

(or failure) surface is controlled by two hardening variables, ϵ_c^{pl} and ϵ_t^{pl} , linked to the failure mechanisms under compression and tension loading, respectively [27]. The damage variables, d_c and d_t are defined together with the hardening variables for stiffness degradation. The zero value of the damage variables defines the initial state of the intact undamaged material. The maximum

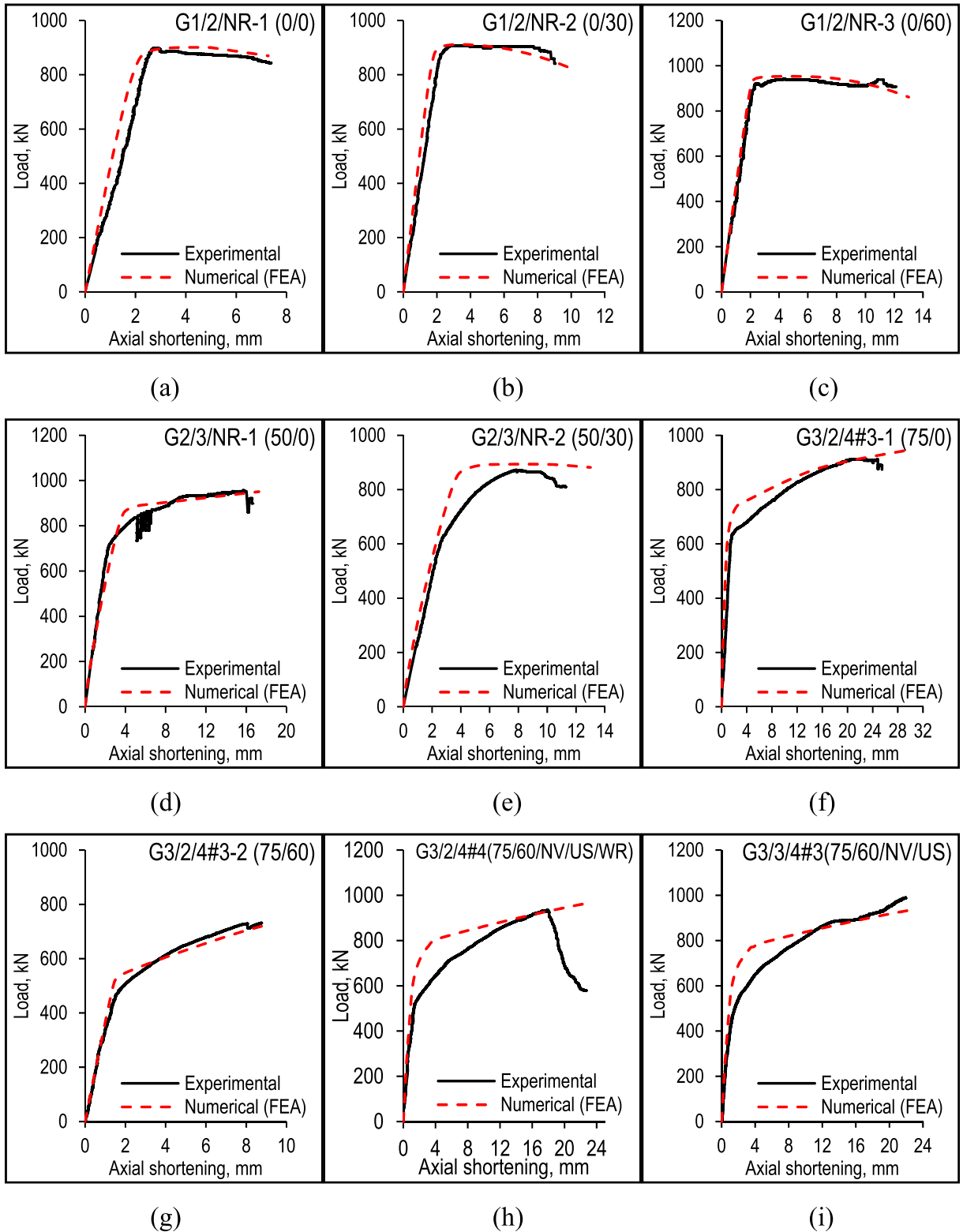
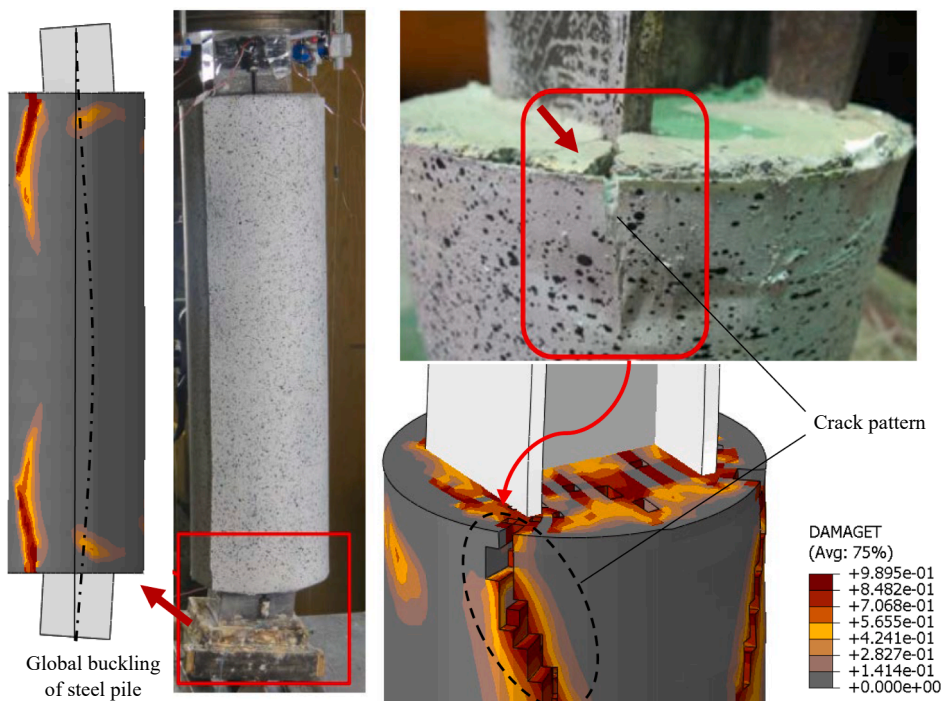


Fig. 9. Comparison of numerical load-displacement curves with those from experiments.

Table 9

Summary comparison of FEA and experimental results.

Designation		Axial Strength (kN)		Axial Strength Exp./FEA	Absorbed Energy (kNmm) ^a		Absorbed Energy Exp./FEA	Exp. HI ^b	FEA HI
		Exp.	FEA		Exp.	FEA			
Group #1	G1/2/NR-1	898.0	901.2	0.996	5006	5341.6	0.937	1.07	1.14
	G1/2/NR-2	907.0	912.0	0.994	5305	5518.4	0.961	1.08	1.05
	G1/2/NR-3	943.0	953.4	0.989	5492	5674.5	0.968	1.08	1.04
Group #2	G2/3/NR-1	956.4	950.8	1.005	4671	4762.4	0.981	1.39	1.14
	G2/3/NR-2	872.3	893.2	0.976	4153	4727	0.879	1.55	1.15
Group #3	G3/2/4#3-1	912.0	944.4	0.965	4248	5070.7	0.838	1.54	1.41
	G3/2/4#3-2	731.0	769.0	0.950	3729	3765	0.990	1.67	1.72
	G3/3/4#3	992.0	934.5	1.061	3996	4798.5	0.833	2.3	1.48
	G3/2/4#4	934.0	962.5	0.970	4035	4937.8	0.817	1.84	1.48
Group #4	G4/2/NR	664.0	661.4	1.003	3127	3365.1	0.929	1.49	1.33
	G4/2/4#4	298.0	657.3	0.453	1656	3194	0.518	1.55	1.36
	G4/3/NR	840.0	865.3	0.970	3258	3982	0.818	2.18	1.94
	G4/3/4#4	943.0	933.3	1.010	3353	3484.2	0.962	2.04	1.67

^a Calculated absorbed energies for loading up to approximately 7 mm axial shortening.^b Hardening index.**Fig. 10.** Comparison of damage modes from FEA and test results (G1/2/NR-1).

theoretical value of these parameters is 1.0, which indicates a complete loss of strength. The typical stress–strain curves of CDP are shown in Fig. 7.

The test specimens in [6] were prepared in four groups at different times. Since four batches of concrete were cast, the concrete properties varied from batch to batch. The compressive strength of the concrete was determined by testing 100x200 mm concrete cylinders according to ASTM C39/C39M-14a [28]. Table 3 summarizes the cylinder test results. The stress–strain relationship of concrete according to CDP model was determined based on the experimentally determined stress–strain data.

In addition to the stress–strain relationships, it is necessary to define several parameters required for the yield and plastic behavior of the CDP model. These parameters related to the plastic flow potential function, and yield function are summarized in Table 4 with brief explanations.

4.2.2. Classical metal plasticity (CMP)

In the CMP model, the elastic and plastic behavior of the material is defined separately. The elastic behavior is defined with elastic modulus, E , and Poisson ratio, ν , while the plastic behavior is defined using a true stress, σ_{true} , and plastic strain, ϵ_{pl} , relationship. The

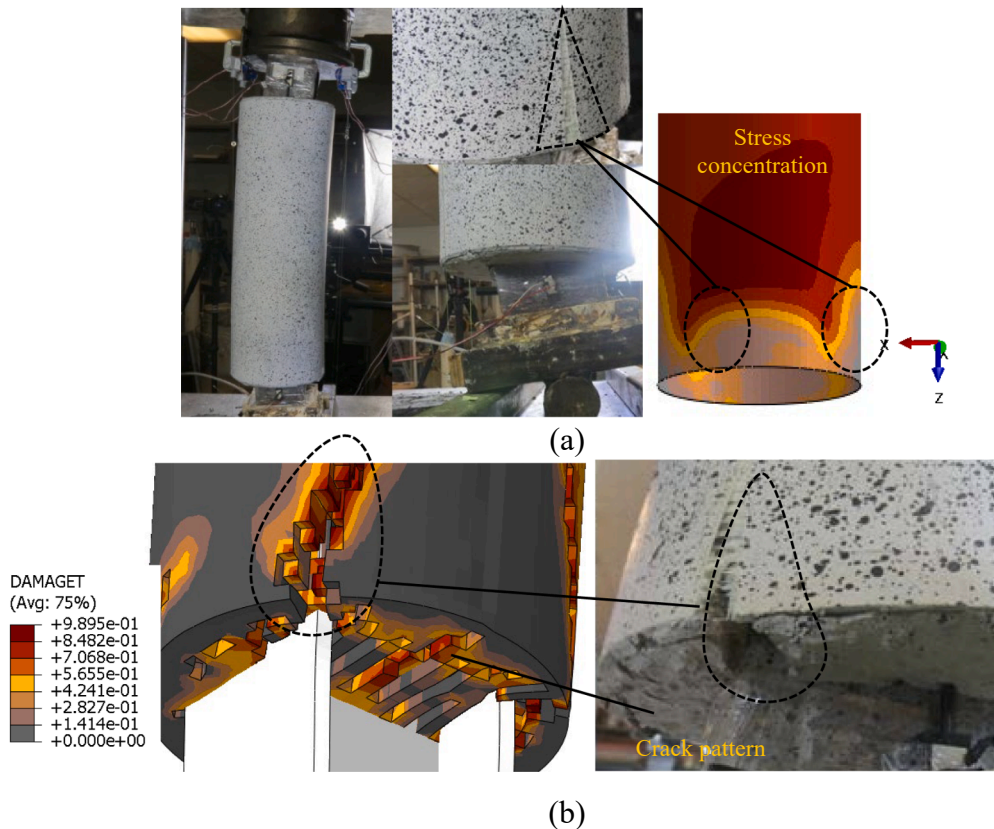


Fig. 11. Comparison of damage modes from FEA and test results (a) G2/3/NR-1, and (b) G2/3/NR-2.

nominal stress and strain data obtained from the tensile tests were converted to a true stress-plastic strain curve and used for modeling the rebar and structural steel. It should also be noted that the von Mises surfaces assume that the yielding of the metal is independent of the equivalent pressure stress in this model. This assumption is confirmed experimentally for most metals under positive pressure stress [38]. Therefore, this material model is suitable for both rebar and structural steel sections. In this research, the material properties of structural steel materials were taken from Karagah et al. [2]. In [2], six tension tests were performed. Two coupons were prepared from the web, and four coupons were prepared from the flanges and tested according to ASTM A370-12a [29]. The coupons were tested under uniaxial tension at displacement rates of 0.6 mm/min and 5.1 mm/min in the elastic and plastic regions, respectively. The summary of tensile test results is given in Table 5 and the mechanical properties of rebar are given in Table 6.

4.2.3. Hashin model

ABAQUS enables the definition of a model that predicts the initiation and evolution of damage for elastic-brittle materials with anisotropic behavior. The model is primarily used for fiber-reinforced materials since they generally exhibit such behavior. The ABAQUS [27] anisotropic damage model is based on the work of Camanho and Davila [30], Hashin [31], Hashin and Rotem [32], and Matzenmiller et al. [33]. In ABAQUS, the damage initiation is determined by the Hashin's theory in [31] and [32]. These criteria are comprehensively discussed in ABAQUS Theory Manual [27]. The parameters required for the FEA of GFRP laminae are given in Table 7. To obtain these material properties, the data from [6] was used. Thin flat GFRP strips were tested according to ASTM D7565M-10 [35] to determine the mechanical material properties of GFRP. Specifically, in [6], two specimens were tested to measure the modulus of elasticity in longitudinal, 0°, and transverse, 90°, directions. In addition, GFRP was tested in the 45° direction to measure the shear modulus. The width and thickness of the GFRP coupons were measured at three locations to determine the cross-sectional area. The GFRP coupons were nominally 147.32 mm long, 25.4-mm-wide, and 0.66 mm thick. Strain gauges were bonded on the face of the GFRP at the center of the coupons to measure the longitudinal and transverse strains. GFRP coupons were tested using a servo-hydraulic machine. The test samples are shown in Fig. 8. The material properties of the adhesive used to bond GFRP jackets to each other, as provided by the manufacturer, are given in Table 8.

5. Results and discussions

A comparison of the load–displacement curves from experiments and FEA is shown in Fig. 9 and the strength values are summarized in Table 9. Group #1 comprised of three test specimens, all of which had reduced a web thickness between 0% and 60% and no

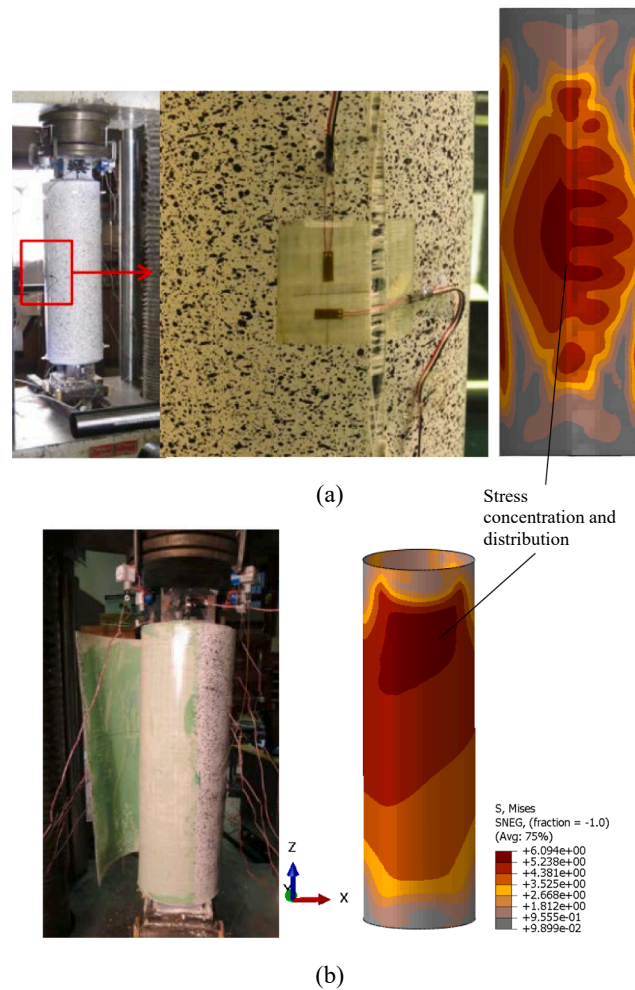


Fig. 12. Comparison of damage modes from FEA and test results (a) G3/2/4#3-1, and (b) G3/2/4#3-2.

reduction of the flange thickness. As shown in Fig. 9, the initial stiffness, until approximately 200 kN load level of the G1/2/NR-1 (0/0), was approximately the same as the experiment. After this load level, the numerically obtained stiffness values were slightly higher, but the difference was negligible. The compressive strength of G1/2/NR-1 (0/0) was obtained as 901.2 kN numerically in comparison to 898 kN experimental value. At 7.36 mm at which the experiment was terminated, the axial strength was 843 kN versus the numerically obtained 880 kN that corresponds to a 4% difference.

As explained in Introduction, experimental and numerical studies were mostly carried out on undamaged samples. However, it is challenging to model a degraded and repaired structural member due to difficulties related to representing the deformed geometry and damaged material properties. Additionally, when it comes to structural repair, many factors affect the result such as workmanship and variability in material properties. In this study, as seen in Fig. 9 and Table 9, the proposed modeling approach has resulted in an accurate prediction of the key characteristics of these damaged and repaired piles. Similar differences between numerical and experimental results were obtained in previous studies [4,39–44].

Similarly, G1/2/NR-2 (0/30) and G1/2/NR-3 (0/60) showed a good agreement in terms of axial strength at the ultimate shortening value. At this stage, the differences between the experimental and numerical strengths were below 4% and 3%, respectively (see Fig. 9 (b) and (c)). When the initial stiffnesses of G1/2/NR-2 (0/30) and G1/2/NR-3 (0/60) is examined, the experimental and numerical values obtained are closer to each other than that for G1/2/NR-1 (0/0). The damage modes obtained numerically for G1/2/NR-1 (0/0) are shown in Fig. 10 as an example for the damage modes of G1/2/NR-2 (0/30) and G1/2/NR-3 (0/60). Concrete cracking, GFRP rupture and global buckling of the steel pile were obtained numerically, and these damage modes are seen to be in good agreement with the experimental observations.

Group #2 included two piles with a 50% reduction of the flange thickness and a 0% or 30% reduction of the web thickness. As seen in Fig. 9 (d) and (e), the transition region from elastic to plastic behavior in the experimentally obtained load-shortening curves had a smaller radius of curvature compared to the numerically obtained curves in this group. This was mainly due to the progressive deformation of the GFRP jacket, and this behavior could not be simulated in the FEA. When the initial stiffness values are examined, an

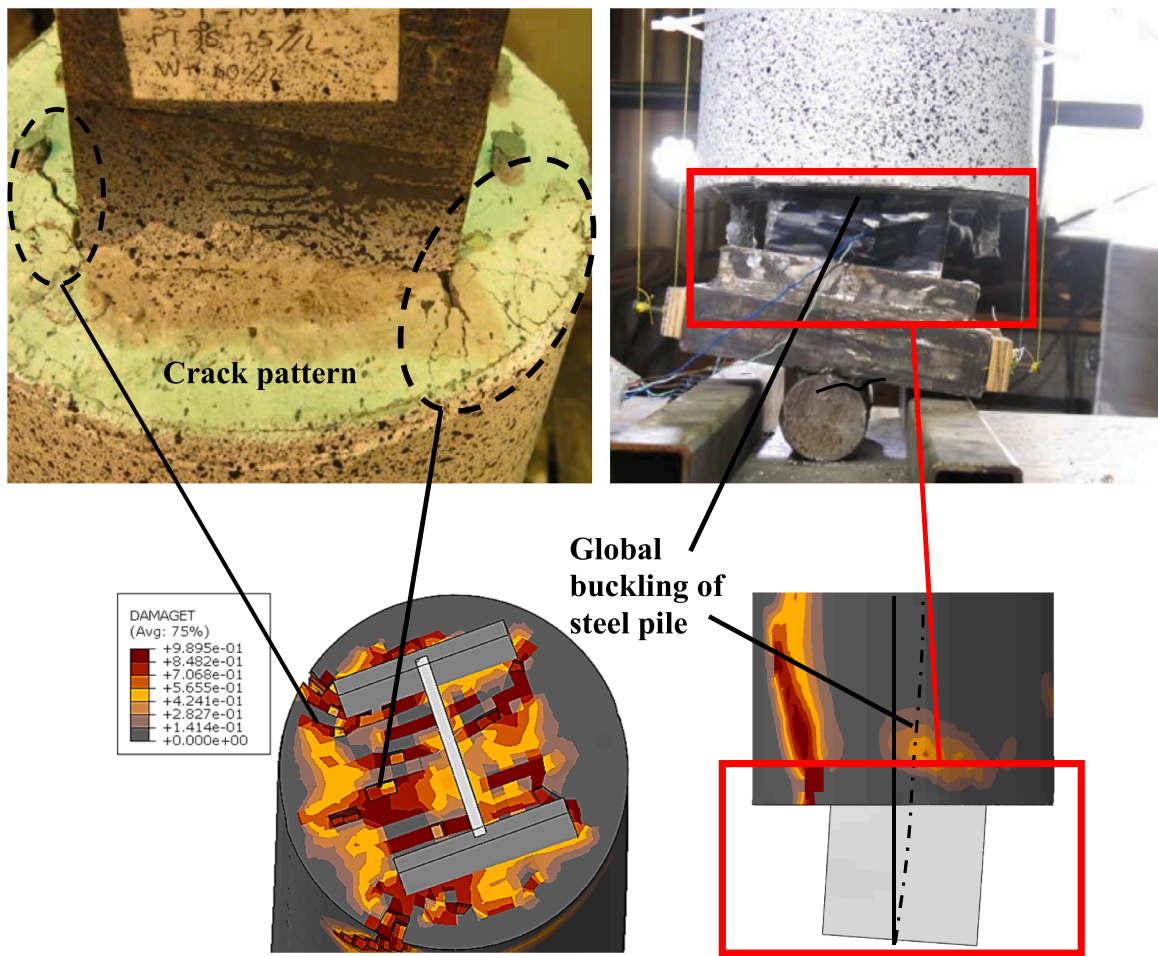


Fig. 13. Comparison of damage modes from FEA and test results for G3/3/4#3.

acceptable agreement between the experimental and the FEA results for G2/3/NR-1 (50/0) are seen. For G2/3/NR-1 (50/0), the value obtained from the numerical analysis was 950.75 kN in comparison to the experimental value of 956.40 kN. When the examination is made for G2/3/NR-2 (50/30), higher strength at the initiation of the plastic region was obtained numerically for this specimen compared to the experiments. The reason for this is that although the location and size of the crack in the concrete agree with the observations in the experimental study, the rupture of the GFRP jacket was not included in the modeling. The axial strength of G2/3/NR-2 (50/30) was obtained as 893.23 kN and 872.3 kN from FEA and experiments, respectively. The load corresponding to the displacement at the end of the testing (11.28 mm) was obtained as 881.9 kN and 808.7 kN from experiments and FEA, respectively, corresponding to a 9% difference. Global buckling of the steel pile and the rupture of the GFRP were observed in the final stages of the experiments. Cracking of the concrete was also observed initiating from the corner of the flange of the steel pile and extending towards the GFRP jacket. This observation in the steel pile and the concrete matched with the damage modes predicted by the FEA as shown in Fig. 11.

Group #3 is comprised of four piles with a 75% reduction of the flange thickness and a 0% or 60% reduction of the web thickness. The strength values obtained from the FEA of G3/2/4#3-1 (75/0) and G3/2/4#3-2 (75/60) were 944.40 kN and 769 kN, respectively (see Fig. 9 (f) and (g)) while the experimentally obtained values were 912 kN and 731 kN. This resulted in 3% and 5% difference with the FEA results respectively for G3/2/4#3-1 (75/0) and G3/2/4#3-2 (75/60). However, when the numerically obtained axial load-shortening curves are examined, the slight decrease in the strength observed in the final stages of the experiments could not be obtained numerically. Although a stress concentration was seen along the height of the GFRP jacket in both the experiments and FEA, the progressive damage and rupture of the GFRP during the final stages of the experiment was not seen in the FEA, see Fig. 12.

The maximum predicted compressive strengths of G3/3/4#3 (75/60/NV/US) and G3/2/4#4 (75/60/NV/US/WR) were 934.46 kN and 962.5 kN, respectively, see Fig. 9 (h) and (i). The values obtained from the experimental study were 992 kN and 934 kN, respectively. For G3/3/4#3 (75/60/NV/US), the difference between the experimental and numerical values was about 6% and the same for G3/2/4#4 (75/60/NV/US/WR) was about 3%. However, when the strength values at the initiation of the plastic region of G3/3/4#3(75/60/NV/US) and G3/2/4#4(75/60/NV/US/WR) are compared, the numerically obtained strength was approximately

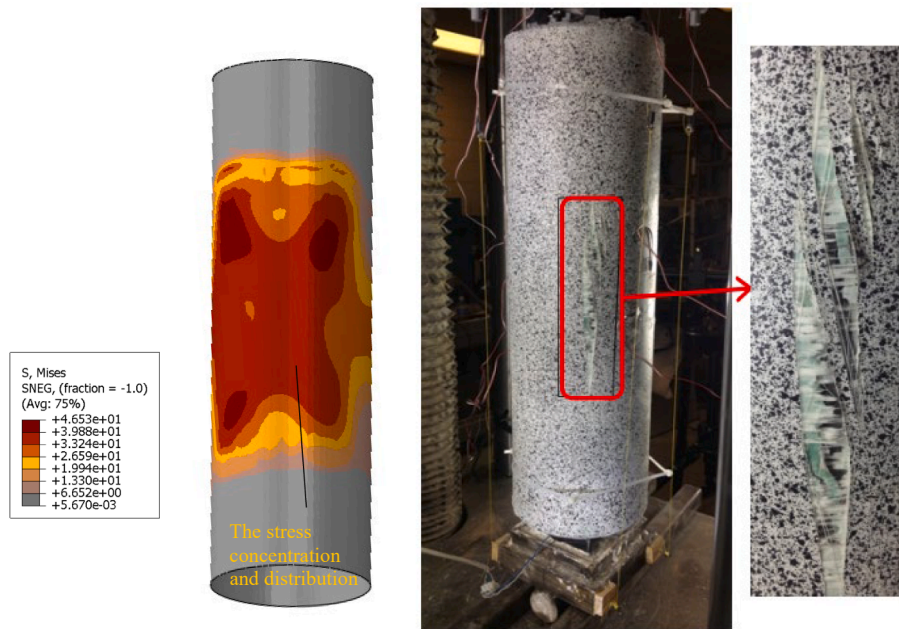


Fig. 14. Comparison of damage modes from FEA and test results for G3/2/4#4.

37% higher than the strength obtained from the experiments. The numerically obtained failure modes for G3/3/4#3 (75/60/NV/US) and G3/2/4#4 (75/60/NV/US/WR) were in good agreement with those obtained from the experimental study, see Fig. 13 and Fig. 14.

Although the numerical study results agreed with the experimental results in terms of axial strength, the sudden strength reduction after reaching the peak could not be replicated in the FEA of G3/2/4#4 (75/60/NV/US/WR). This is because the damage and GFRP rupture in the final stages of the experiment were not included in the FEA although stress concentrations were observed, see Fig. 14.

Group #4 included four piles with a 75% reduction of the flange thickness, 60% reduction of the web thickness, and a void in the web to simulate through-corrosion. As seen in Fig. 9 (j), (k), (l) and (m), the initial stiffness obtained from the FEA for this group of piles was in good agreement with the experimental results. The difference between the numerical and experimentally obtained axial strength values was negligible for G4/2/NR (75/60/V/S). Like the FEA results from Group #3, when the strength values at the initiation of the plastic region is compared, the numerically obtained result was approximately 15% higher than that obtained from the experiments. Similar to the other test specimens, global buckling of the steel pile and the rupture of the GFRP jacket were observed as shown in Fig. 15.

Testing of pile G4/2/4#4 showed that the repair was not correctly done. Notably, during concrete casting, the concrete began to set prior to filling the GFRP jacket (Fig. 16). Consequently, there were substantial air voids within the jacket. Despite this observed weakness during the repair process, the pile was tested to see how incomplete filling affected the repaired pile's performance. Because of this problem, the increase in capacity of the repaired pile was less than expected.

As seen in Table 9, the pile had a significantly lower capacity than the other three repaired piles in the same group. Fig. 9 (j) shows that the plateau in the plastic region seen in the experimentally obtained axial load-shortening curves could not be obtained numerically as this was not incorporated in the FEA. For G4/3/NR (75/60/V/US) and G4/3/4#4 (75/60/V/US/WR), the differences between the numerical and experimental axial strengths were about 3% and 1%, respectively (see Table 9). However, the experimentally obtained strength degradation could not be obtained in the last part of the axial load-shortening curve for G4/3/NR (75/60/V/US). The rupture of the GFRP jacket during the final stages of the experiment was not seen in the FEA like the results in Groups #1-#3. As seen in Fig. 9 (l), the initial stiffness from the FEAs was in good agreement with the experimental results. The experimentally obtained axial strengths are compared to numerically obtained values in Fig. 17 where the 45-degree line represents a perfect match. It is seen that the data points cluster along the 45-degree diagonal line, which indicates a good agreement between the FEA and the experimental results.

The absorbed energy is important in evaluating the nonlinear behavior of materials, structural elements, and, thus, the overall system. In this study, besides comparing the experimental and numerically obtained load-bearing capacities, the energy absorption capacities of the steel piles after strengthening were calculated using the area under the load-axial shortening curves. To make a fair comparison between all the piles in different groups, the minimum ultimate axial shortening value of approximately 7.0 mm for all groups was considered, and the energy absorption values as a function of axial shortening is shown in Fig. 18. The energy absorption values for Group #1 were higher than all the other piles in other groups. Additionally, in Group #1, when the section loss increased from 0 to 16.1%, the energy absorption increased by approximately 9%. In Group #2, energy absorption was approximately 12% higher in the case of cross-section reduction in the flanges only (G2/3/NR-1) than in the flanges and the web (G2/3/NR-2). In Group

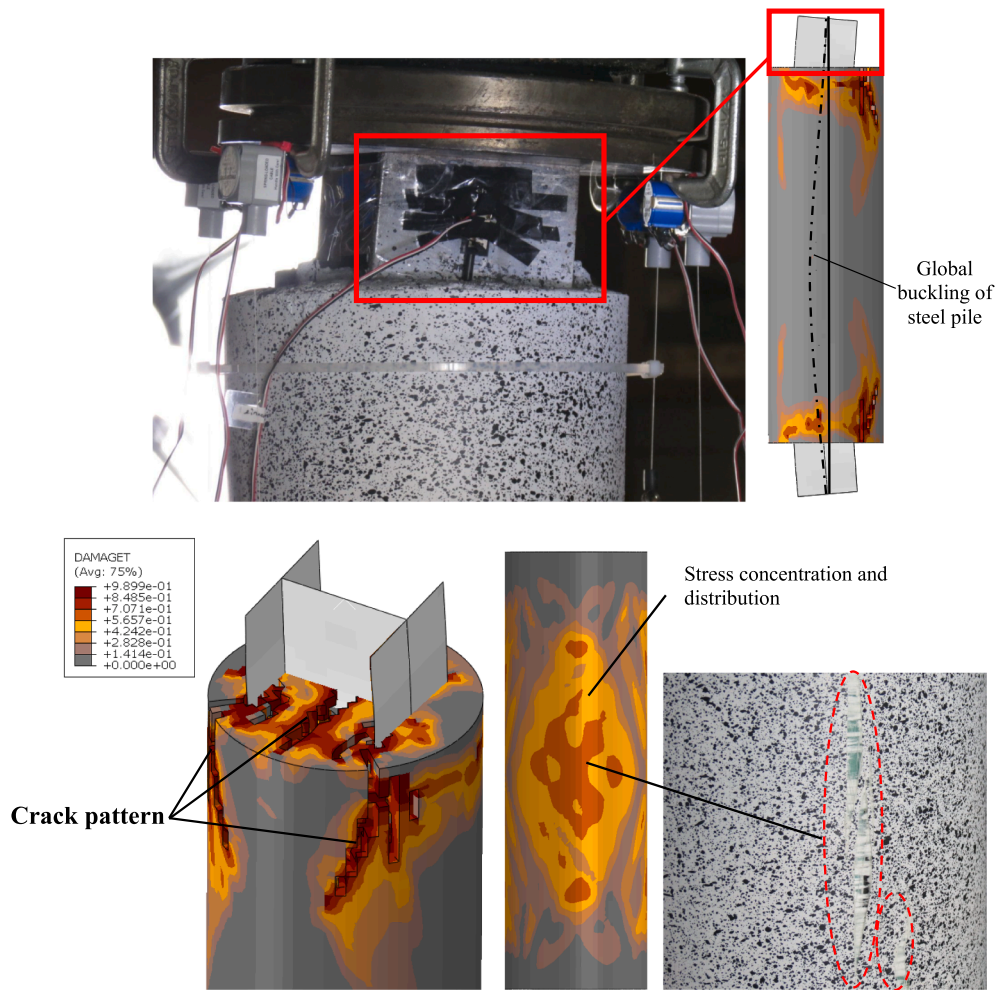


Fig. 15. Comparison of damage modes from FEA and test results for G4/2/NR.

#3, G3/2/4#3-1, with a 75% cross-section reduction only in the flanges, had a higher energy absorption than the other elements in this group. Like Group #2, the flange-only cross-section reduction had approximately 14% higher the energy absorption capacity than the cross-section reduction in both flanges and the web, i.e., G3/2/4#3-2.

Further, when considering the energy absorption capacities obtained for other members of Group #3 without a reduction in the flange width (i.e., G3/2/4#4 and G3/3/4#3), the difference between the energy absorption capacity were found to be negligible. In addition, when the energy absorption values obtained for Group #4 are examined, the results showed that G4/2/4#4 had a lower energy absorption capacity than the other group members. This is attributed to the construction error during concrete casting (see Fig. 16). Because of this problem, the energy absorption capacity of the repaired pile was less than expected. The difference between other group members was less than 7%. The results showed that absorbed energy values were predicted with an acceptable error, except for that for G4/2/4#4 (see Fig. 19).

The hardening index (HI) is another crucial parameter that measures a material or a member's ability to redistribute the load and maintain stability under increasing displacement demands. It is defined as the ratio of a member's load-bearing capacity after plastic deformation to its yield strength. A high HI indicates that a structural member can withstand increasing loads after yielding and before failure. It also determines a structure's overall stability under increasing loading. In Table 9, both experimental and numerical hardening indexes of corroded and buckled steel piles after strengthening are presented. These results showed that except for in three cases (G2/3/NR-2, G3/3/4#3 and G4/3/4#4), the computational simulations accurately predicted the HI of the tested piles.

6. Parametric study

The parametric study presented in this section is aimed to investigate the effectiveness of the proposed strengthened technique using the validated numerical models presented above. Specifically, the different concrete strengths or losses of cross-sectional dimensions of the piles, which could not be studied experimentally due to limitations in resources, are evaluated on the reference



Fig. 16. Construction error during concrete casting of G4/2/4#4.

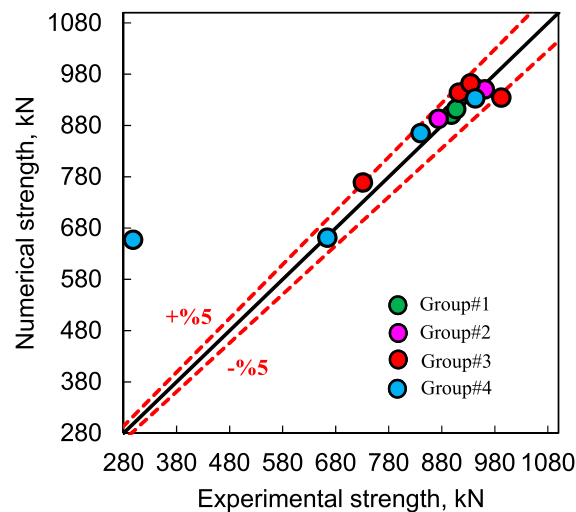


Fig. 17. Comparison of experimental and numerical axial strength results.

specimen, G1/2/NR-1. The geometry was varied by changing one parameter at a time to create a new model. The flange and the web thicknesses were reduced from 10% to 70% in 10% increments either individually or together and the load-axial shortening curves were obtained as shown in Fig. 20. Fig. 20 also shows that the unbuckled and non-corroded pile (0/0) has the highest initial stiffness and compressive strength. A sudden decrease in strength is observed after the peak load because the buckling of 0/0 was not restrained. Additionally, concrete strength classes of C12/15, C30/37, C70/85 and C90/105, were evaluated and load-axial shortening curves were obtained as shown in Fig. 21.

As expected, the maximum load-bearing capacity was obtained from the C90/105 concrete. When the compressive strength increased by about 80% compared to the reference test specimen, the increase in axial compressive strength was about 4%. These findings show that concrete compressive strength plays a secondary role in the maximum load-bearing capacity of strengthened steel piles. This is because concrete is not used as a load-bearing element but to restrain the transverse displacement of the steel pile under load. When the effect of section loss due to corrosion along the height of the steel pile was examined, the results showed that, as expected, section loss causes a decrease in the axial load-bearing capacity of the strengthened steel pile. Especially in the case of 50% to 70% thickness reduction in the flanges and the web, local buckling occurred in the flanges and the web before further damage to the concrete and GFRP (see Fig. 22). In this case, a significant softening branch was formed without hardening in the load-axial shortening curves (see Fig. 20 (a)). This response is undesirable due to limited load redistribution and overall system instability. Additionally, as

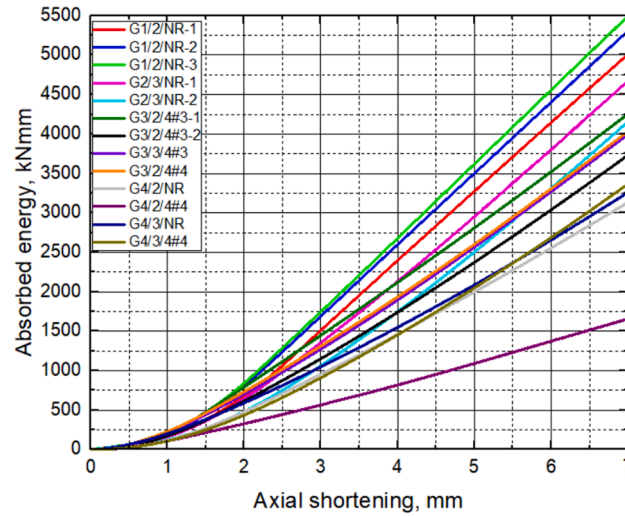


Fig. 18. Absorbed energy-axial shortening curves.

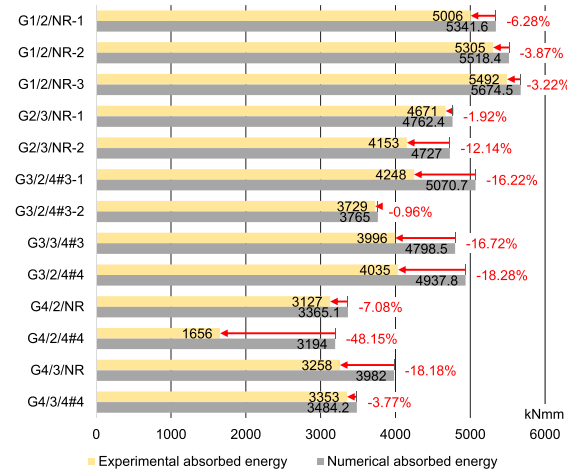


Fig. 19. Comparison of experimental and numerical absorbed energy.

seen in Fig. 20 (b) and (c), when the effects of section loss due to corrosion on the flanges and the web on the axial load-bearing capacity of the strengthened steel pile is examined separately, in the case of thickness reduction of 70% in the flanges or the web, the strengths decreased by 52% or 20%, respectively, compared to the reference specimen. These results showed that the decrease in the thickness of the flanges is more detrimental than the decrease in the thickness of the web.

When an evaluation is made regarding the absorbed energy values, as seen in Table 10, the lowest value was obtained in the case of 70% thickness reduction in both the flanges and the web, like what was observed for the load-bearing capacities. In addition, the piles with the lowest amount of reduction in absorbed energies compared to the control element were only the elements with a reduced web thickness. For this group, the difference between the absorbed energy values between the 10% and 70% thickness reduction cases was approximately 17%. Moreover, in Fig. 23, scatter plots present relationships between normalized cross-sectional area and compressive strength or absorbed energy. The normalized cross-sectional area expresses the ratio of the reduced area, A_r , due to corrosion to the steel pile's gross cross-sectional area, A_g . It is seen in the figures that there is approximately a linear relationship between both the compressive strength and the absorbed energy values and the normalized cross-sectional areas. These parametric studies showed that in the case of strengthening with grout and GFRP layers, the load bearing and energy absorption capacity of strengthened members can be approximately predicted by the A_r/A_g ratio, and the axial compressive strength or absorbed energy obtained from the control element either using an experiment or a validated computer model as shown in this paper.

7. Summary and conclusions

This paper discusses the findings and observations of the effectiveness of a GFRP-based system for rapid retrofit of buckled steel

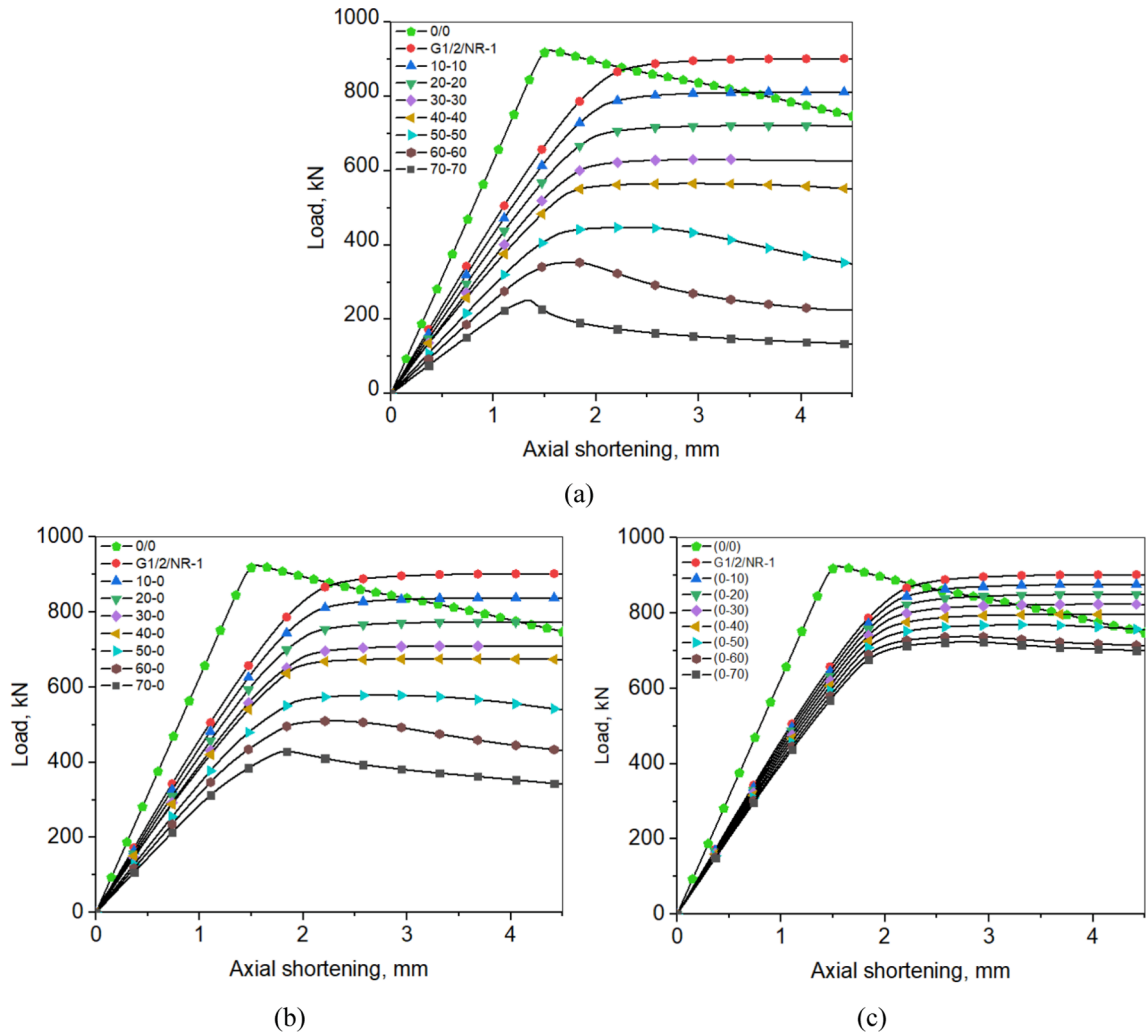


Fig. 20. Parametric study results for cross-sectional dimensions: (a) thickness reduction in flanges and web, (b) thickness reduction in flanges only, and (c) thickness reduction in web only.

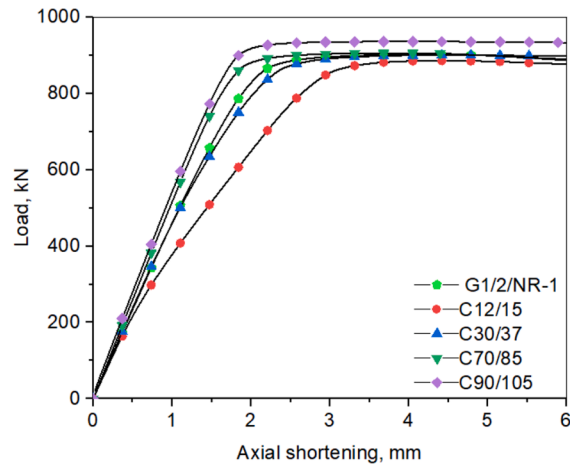


Fig. 21. Parametric study results for concrete strength.

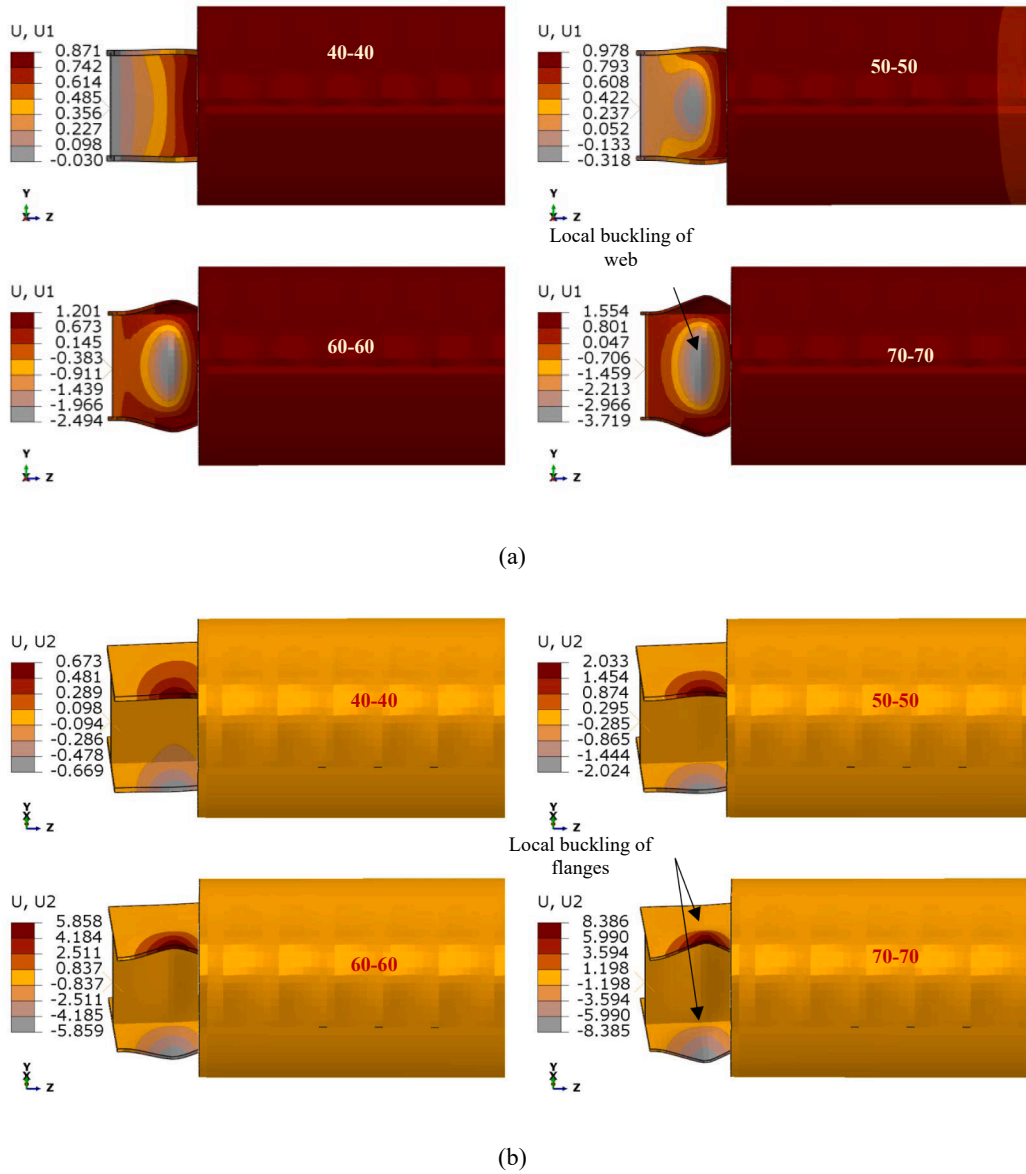


Fig. 22. Local buckling of (a) web and (b) flanges of steel pile under cross-sectional reduction due to corrosion.

piles with corrosion damage. A new modeling approach has been developed and non-linear FEAs were conducted to investigate the remaining axial compression capacity and damage modes of deteriorated steel piles retrofitted with concrete and GFRP. Based on the findings, the following conclusions are drawn.

- It was shown that the proposed modeling approach could be used with acceptable accuracy, especially when an evaluation is made in terms of maximum axial strength. It was seen that the proposed method correctly predicted the location and size of damage and deformation modes in the concrete and the steel pile. The stress concentrations on the GFRP jacket were also accurately estimated.
- When the axial load-shortening curve obtained from both experiments and FEA was examined, a hardening response was observed for most of the strengthened piles in the non-linear range. Corroded but unrepaired steel piles exhibited a softening response after ultimate load-bearing capacity. A hardening response is desired in that it ensures load redistribution and overall system stability.
- In the proposed repair method, there is a linear relationship between the number of GFRP layers and the ultimate load-bearing capacity of the specimens. When comparing samples with similar axial load capacity, the FEA result showed that three layers of GFRP was 30% more effective than two layers of GFRP. The experimentally obtained difference was 27%. This shows that FEAs can predict the relationship between the number of GFRP layers and ultimate load-bearing capacity with an acceptable accuracy.

Table 10
Summary of parametric study results.

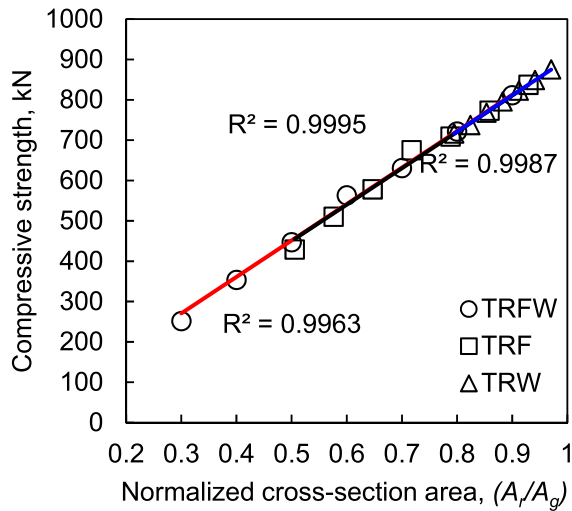
Designation	Thickness Reduction in Web or Flanges (%)	Retrofitting Scheme	No of GFRP Layers	Compressive Strength (kN)	Absorbed Energy (kJ mm) ^a
TRFW ^b	10–10	Grout + GFRP	2	811.5	2851.4
	20–20			721.4	2560.7
	30–30			630.7	2268.2
	40–40			563.3	2057.8
	50–50			447.3	1567.2
	60–60			353.8	1114.5
	70–70			251.8	700.1
TRF ^c	10–0	Grout + GFRP	2	837.5	2931.4
	20–0			773.4	2723.0
	30–0			709.0	2518.1
	40–0			675.2	2428.4
	50–0			578.2	2073.3
	60–0			510.3	1785.9
	70–0			428.9	1466.5
TRW ^d	0–10	Grout + GFRP	2	875.3	3050.3
	0–20			849.2	2970.4
	0–30			822.9	2889.4
	0–40			796.2	2800.3
	0–50			768.4	2716.5
	0–60			737.8	2608.3
	0–70			716.3	2532.3

^a Absorbed energy calculated up to 4.5 mm axial shortening.

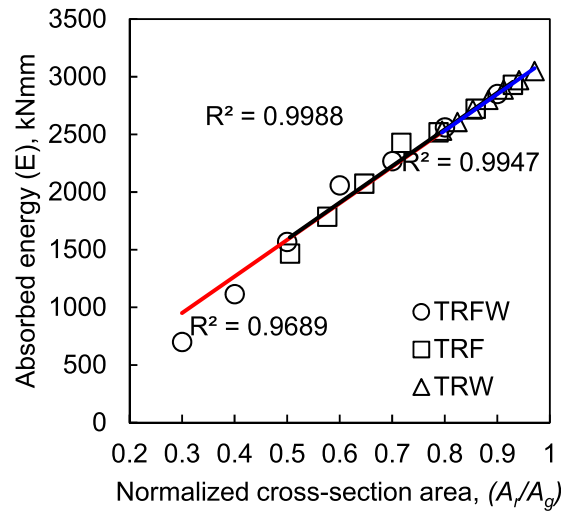
^b Thickness reduction in both flange and web.

^c Thickness reduction only in flange.

^d Thickness reduction only in web.



(a)



(b)

Fig. 23. Relationships between normalized cross-sectional area and (a) compressive strength, and (b) absorbed energy. TRFW: thickness reduction in both flange and web, TRF: thickness reduction only in flange, TRW: thickness reduction only in web.

- The addition of internal reinforcing bars helped increase capacity by up to 8% in the FEA and up to 12% in the experimental study. Specimens with a 12.7 mm and 9.5 mm diameter rebar exhibited similar behavior and capacity. The effect of rebar could be predicted with acceptable accuracy using the developed models.
- The effectiveness of the repair technique is highly sensitive to the quality of application and installation. This situation directly concerns engineers and practitioners. For example, the G4/2/4#4 specimen had a significantly lower capacity than the other three repaired piles in the same group. The plastic plateau seen in the experimental axial load-shortening curve for this specimen could

not be obtained numerically. This has clearly shown that one of the most critical steps during the repair process is the concrete casting.

- The parametric study results showed that the concrete compressive strength plays a marginal role in the maximum load-bearing capacity of the strengthened steel piles. Moreover, in the case of 50% to 70% thickness reduction in the flanges and the web, local buckling occurred before further damage to the concrete and GFRP. This is undesirable because of a lack of load redistribution and because it leads to overall system instability. It was also found that a decrease in the thickness of the flanges is more detrimental to the load-bearing capacity of the strengthened piles than a decrease in the thickness of the web.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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