

# Non-linear finite element analysis of grouted connections for offshore monopile wind turbines

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12 **- - Abstract - -**

13 Grouted Connections (GCs) are vital structural components of Offshore Wind Turbine (OWT)  
14 substructures. On monopiles to achieve a GC, tubular hollow steel piles are in-situ attached with a high-  
15 strength grout. Monopiles are susceptible to large magnitude bending loads in offshore environments.  
16 Recently, following inspections the performance of GCs has been called into doubt when settlements  
17 were reported on several monopiles. To further comprehend the structural performance of GCs under  
18 large bending moments a nonlinear Finite Element (FE) analysis was conducted. Three-dimensional FE  
19 models were solved and validated against experimental and analytical data with good agreement. It is  
20 suggested that the presented models can be used to evaluate the global and local behaviour of a GC  
21 accurately. Finally, a comprehensive parametric study was carried out to investigate the influence of  
22 shear key numbers, shear key spacing and overlap lengths. It was shown that increased number of shear  
23 keys are advantageous for stiffness and reduce the gap at the interfaces, whereas the grout failure  
24 depends on the spacing between neighbouring shear keys. The ability of the numerical model to trace all  
25 relevant failure modes which are provoked by shear key spacing was also demonstrated.

26 **Keywords:** *Offshore Wind Turbines; Finite Element Analysis; Grouted Connections; Monopile; High-*  
27 *strength grout; Shear keys*

28 **Abbreviations**

29	FE	<b>Finite Element</b>
30	GC	<b>Grouted Connection</b>
31	O&G	<b>Oil and Gas</b>
32	OWT	<b>Offshore Wind Turbines</b>
33	PGC	<b>Plain Grouted Connection</b>

34	SKGC	<b>G</b> ROUTED <b>C</b> ONNECTION WITH <b>S</b> HEAR <b>K</b> EYS
35	HSG	<b>H</b> IGH <b>S</b> TRENGTH <b>G</b> ROUT
36	CDP	<b>C</b> ONCRETE <b>D</b> AMAGE <b>P</b> LASTICITY
37	TP	<b>T</b> RANSITION <b>P</b> IECE
38		

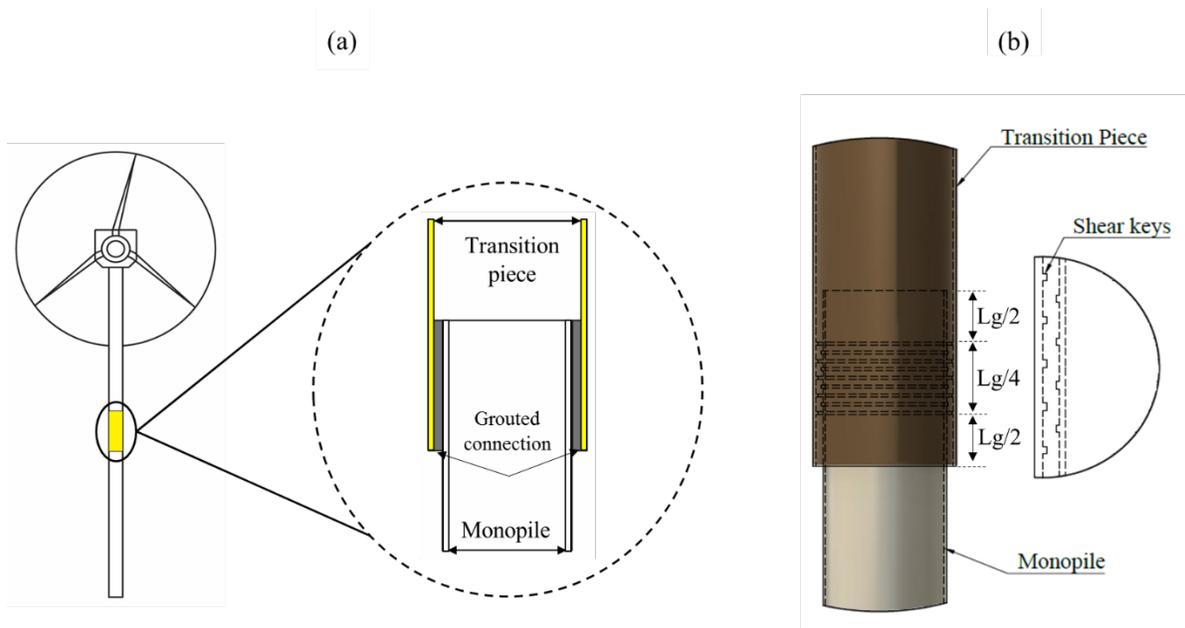
## 39 **1. Introduction**

40 Grouted Connections (GCs) are particularly favoured in offshore structures and their use has been  
41 common practice on Offshore Wind Turbines (OWTs) over the last decades. Their robustness has been  
42 proven within the Oil and Gas (O&G) sector, mainly when connecting piles with jacket legs or  
43 strengthening structural parts of offshore platforms [1]. On monopile OWTs, GCs are achieved by in-  
44 situ filling the annuli between overlapping tubular steel shells with a High-Strength cementitious Grout  
45 (HSG) (figure 1a). The grout attaches the monopile to the transition piece (TP), while allowing the  
46 transition from the substructure to the tower. It also ensures the vertical alignment of the latter,  
47 alleviating possible inclinations during installation. Owing to their arrangement, GCs are often referred  
48 to as pile to sleeve connections in the literature.

49 GCs on OWTs are based on the same principal with existing connections used in O&G structures.  
50 However, monopiles are characterised by large diameter-to-thickness ratios and the loading regime  
51 acting upon the substructure differs. Bending moments are the dominant effect on a monopile caused by  
52 the combined wind and waves action [2, 3]. In the early days of OWTs, inspections on monopiles  
53 revealed unexpected settlements of the TP on several turbines in Europe [1, 4]. The large magnitude  
54 bending loads on a GC result in high tensile stresses on the grout, which subsequently induces cracking  
55 in different directions. Furthermore, the ovalisation of the steel piles leads to a gap being formed at the  
56 steel grout interface [5]. As a result, the interface gaps lead to water ingress, which was also reported on  
57 some of the inspected monopiles [6].

58 Consequently, there has been increasing research interest on GCs aiming to enhance the design process.  
59 Particular attention is on monopiles due to the scale of the substructure and the scarce test data on large-  
60 diameter connections. Experimental campaigns have focused on data generation for GCs [see, e.g., 2, 3,  
61 7] to comprehend the reasons that caused the unexpected slippages of the TPs on plain pipe connections.

62 Findings from these studies suggested the use of welded beads on the circumference of the tubular  
 63 shells, known as shear keys, to provide mechanical interlock. Shear key types include semi-circular  
 64 welded beads or fillet-welded square bars which are commonly fabricated within a central region of the  
 65 connection as shown in figure 1b. As a result, design guidelines for GCs [8, 9] have been revised aiming  
 66 to provide further assistance on the design of GCs with shear keys. Recently tests on GCs have focused  
 67 on the fatigue performance of GCs [10]. To date, the use of tubular steel sections with circumferential  
 68 shear keys or conical steel tubes without shear keys is the common approach for OWTs. However, as the  
 69 connections are affected by numerous geometrical parameters (e.g., shear keys, grouted length, radial  
 70 stiffness etc.) and environmental factors, further studies are needed to bridge the gap of knowledge on  
 71 their structural performance.



72

73 **Figure 1:** a) Layout of monopile with GC (after modification [4]) and b) GC with shear keys, where  $Lg$   
 74 refers to the grouted length

75 Apart from physical modelling, numerical methods are a promising alternative which compensate for the  
 76 expenses of experimental campaigns. Finite Element (FE) analysis of GCs can be of remarkable benefit

77 during the design process. It can provide a detailed insight on the grout condition as well as the load  
78 transfer mechanisms taking place in the connection. Additionally, parametric studies enable the  
79 investigation of numerous parameters once a validated model is achieved. Still, modelling of GCs is an  
80 intensive process when it comes to computational resources. This is due to the distinct brittle nature of  
81 the HSG, along with the interface modelling of steel, grout and shear key inclusion. Past studies [see,  
82 e.g., 7, 11, 12] tackled the intensity of the computations using numerical approaches of varying  
83 complexity.

84 A first attempt to provide general considerations for the numerical analysis of GCs is made by Nielsen  
85 [13]. A review of the available constitutive models for concrete-based material is presented along with a  
86 discussion on contact formulations. The degradation of the grout due to cyclic loading was not  
87 considered a major concern back then, as the consensus was that a HSG would compensate for those  
88 actions. The inclusion of shear keys was also not discussed as the practice was the installation of plain  
89 pipe connections. However, the sliding of TPs dictated the use of circumferentially-welded shear  
90 connectors and prompted researchers to include them in numerical analyses. Shear keys are usually a  
91 few millimetres in size which often leads to models with many elements requiring significant  
92 computational resources. Fehling et al. [11] and Löhning and Muurholm [14] discussed alternative  
93 techniques to compensate for the additional computational effort induced from the inclusion of shear  
94 keys. Those involve the representation of shear keys with springs acting diagonally or vertically. A  
95 similar approach was employed by Wilke [7] for shear key representation. This method reduces the  
96 computational cost, however employing such an approach introduces a level of uncertainty as it requires  
97 the calibration of the spring stiffness with experimental data. Furthermore, it was shown that because of  
98 the de-bonding occurring at the interfaces it cannot be applied to regions where the shear keys lie close  
99 to the opening of the interface. In the case of monopiles gap opening between the steel and grout can be

100 accounted as one of the reasons for insufficient performance and can develop significantly along the  
101 length of the connection.

102 Computational demand is also affected from the selected constitutive material models. Effective  
103 modelling of HSG requires the inclusion of cracking and crushing behaviour. Wang et al. [15]  
104 investigated the axial capacity of GCs using the brittle cracking model, whereas Andersen and Petersen  
105 [16] and [7] used the Drucker-Prager model to define the HSG. In the latter study is noted that the  
106 overall response of the connection was overestimated owing to the softening behaviour of the grout  
107 being suppressed. [14] employed the Concrete Damage Plasticity (CDP) model to describe the grout  
108 behaviour of a plain pipe GC. The FE study focused on the interaction of bending and axial loads,  
109 however the model was only calibrated against an axially-loaded small scale GC without considering  
110 scale effects and differences in loading configuration.

111 Considering past studies [7, 11, 12] it was shown that the brittle nature of the grout, the interface  
112 modelling and the inclusion of shear keys, makes modelling of GCs a complex process. To date there is  
113 a lack of numerical studies with validated models focusing on the flexural behaviour of GCs with shear  
114 keys. This paper aims to present a consistent methodology to develop and validate three-dimensional  
115 numerical models of GCs, which can subsequently predict the failure modes that were found to be  
116 present on monopiles. Herein, in section 2, the experimental tests on down-scaled GCs which were used  
117 in this numerical study are briefly presented. Section 3 addresses the model specifics, such as mesh  
118 discretisation, material modelling and boundary conditions. Section 4, outlines the validation study  
119 conducted, while section 5 focuses on the parametric study to enhance the body of knowledge on the  
120 design of GC using FE analysis. Finally, in section 6 the conclusions of this work are presented.

## 121 **2. Experimental tests on GCs for monopiles**

122 In this study the experiments presented in [2, 7] are used to validate the numerical models. For this  
123 purpose, two GCs of approximately 1:6 scale were employed. The connections were loaded under a 4-  
124 point bending configuration. The tests were selected due to the documented failure modes being similar  
125 to those that were reported during the inspections of in-service monopiles. Experimental failure modes  
126 included grout cracking in the vicinity of shear keys, gap at the interface and deformation of the steel  
127 pile. The selected tests involve a plain pipe connection (PGC) and one with shear keys (SKGC). The  
128 dimensions of the GCs are given in Table 1, along with the DNV [9] recommended limits of application.  
129 Within Table 1 a non-dimensional parameter, the overlap length ( $F_o$ ), is introduced. It is defined as the  
130 ratio of the grouted length over the pile diameter and is often used to compare GCs. It is worth noting  
131 that the examined GCs are of a lower overlap length than the limit suggested in [9].

### 132 **3. FE modelling and numerical scheme**

133 The following sections present the developed FE models and validation against the experimental data.  
134 For all the subsequent analyses the general-purpose FE software ABAQUS [17] was employed and a  
135 high-performance computational cluster was used for the computations. Three-dimensional models  
136 including geometrical and material non-linearity were solved by means of a quasi-static explicit  
137 analysis. The explicit method is a dynamic process, however when highly non-linear material behaviour  
138 and contact interactions are involved, it is an effective alternative to address convergence issues that  
139 often arise. This numerical approach has been effectively employed in previous studies, in a variety of  
140 structures involving concrete and interface problems [18, 19]. For the presented models, the  
141 computational cost was sufficiently reduced by using semi-automatic mass scaling with a fixed time  
142 increment of 4.5E-6 s in every step. A sensitivity analysis was performed to select the time increment,  
143 aiming to achieve a quasi-static solution by maintaining negligible inertia effects and artificial strain  
144 energy.



**Table 1.** Dimensions of FE models

<b>Description</b>	<b>Symbol</b>	<b>Value<sup>1</sup></b>	<b>Limit<sup>2</sup></b>
Shear key height [mm]	$h$	3	n/a
Shear key spacing [mm]	$s$	60	n/a
Shear key width [mm]	$w$	6	n/a
**Shear key ratio [-]	$h/s$	0.05	$< 0.1$
**Width to height ratio [-]	$w/h$	2	$1.5 < w/h < 3$
Shear key number [-]	$n$	7	
Pile, Sleeve length [mm]	$L_P, L_S$	1955	n/a
Grout length [mm]	$L_g$	1040	n/a
Pile diameter, thickness [mm]	$D_P, t_P$	800, 8	$10 < R_P/t_P < 30$
Sleeve diameter, thickness [mm]	$D_S, t_S$	856, 8	$9 < R_{TP}/t_{TP} < 70$
Grout diameter, thickness [mm]	$D_g, t_g$	840, 20	n/a
Overlap length [-]	$F_o = L_g/D_P$	1.3	$1.5 < L_g/D_P < 3$
End-beam length, thickness [mm]	$L_B, t_B$	1950, 14	n/a

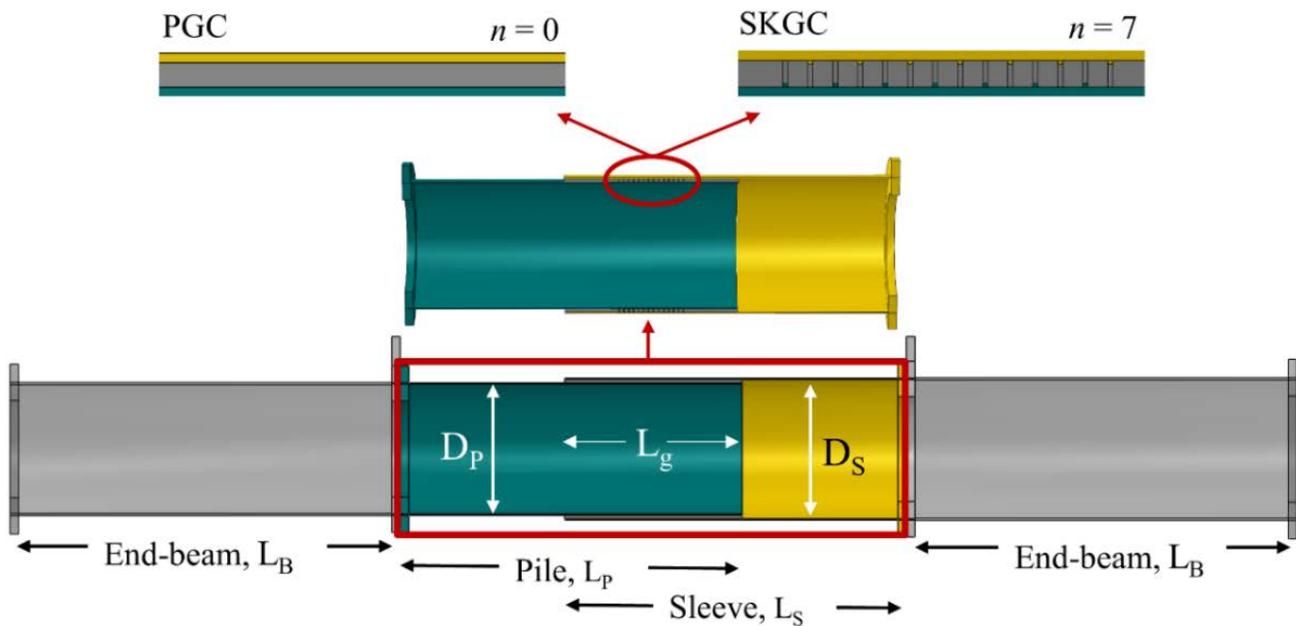
147

<sup>1</sup>Wilke (2013),<sup>2</sup>DNV (2014), \*\* Applies only to the SKGC model

### 148 3.1 Model Geometry and Discretisation

149 A schematic representation of the FE models which are developed for the validation study is given in  
150 figure 2. The model geometry is developed to be identical to the experimental set-up. Throughout the  
151 models, 8-node solid elements with reduced integration (C3D8R) were used to discretise the grout and  
152 steel. Along the thickness of all parts a minimum of three elements were used. The end-beams were  
153 meshed with a larger element size compared to the pile, sleeve and grout to reduce the total number of

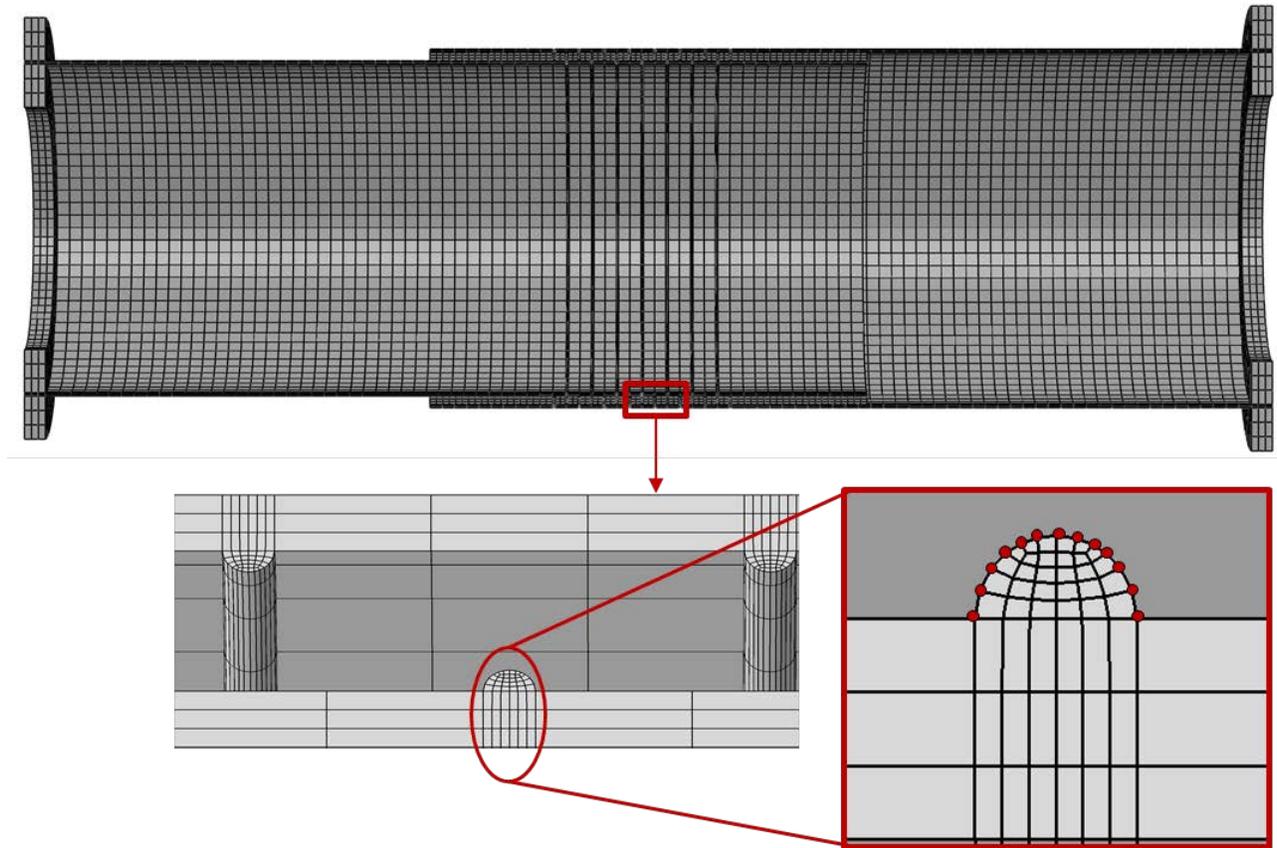
154 elements. However, the size of the shear keys in the SKGC model led to an increased number of  
155 elements and a higher-density mesh on the grout was required. In order to appreciate the effect of shear  
156 keys on GC discretisation, on the PGC model approximately 37,000 elements were employed, whereas  
157 for the SKGC model a total of 93,042 elements. The shear keys were modelled as perfectly circular  
158 beads without considering welding irregularities to achieve higher mesh quality. For refinement  
159 purposes twelve elements were used along the circumference of each shear key as shown in figure 3, to  
160 achieve a perfectly-circular shear key geometry. Solution time is dictated from the element size in  
161 explicit computations, hence the SKGC model proved to be computationally-demanding when compared  
162 to the PGC as expected.



163

164

**Figure 2.** FE model definition of PGC and SKGC



165  
166

**Figure 3.** Meshed connection and discretisation around the cross section of a semi-circular shear key

167

### 3.2 Boundary conditions, constraints and interactions

168

Both PGC and SKGC were equipped with the same constraints and interaction properties for

169

consistency. Parts of the test-rig and the spreader beam were not included in the numerical models and

170

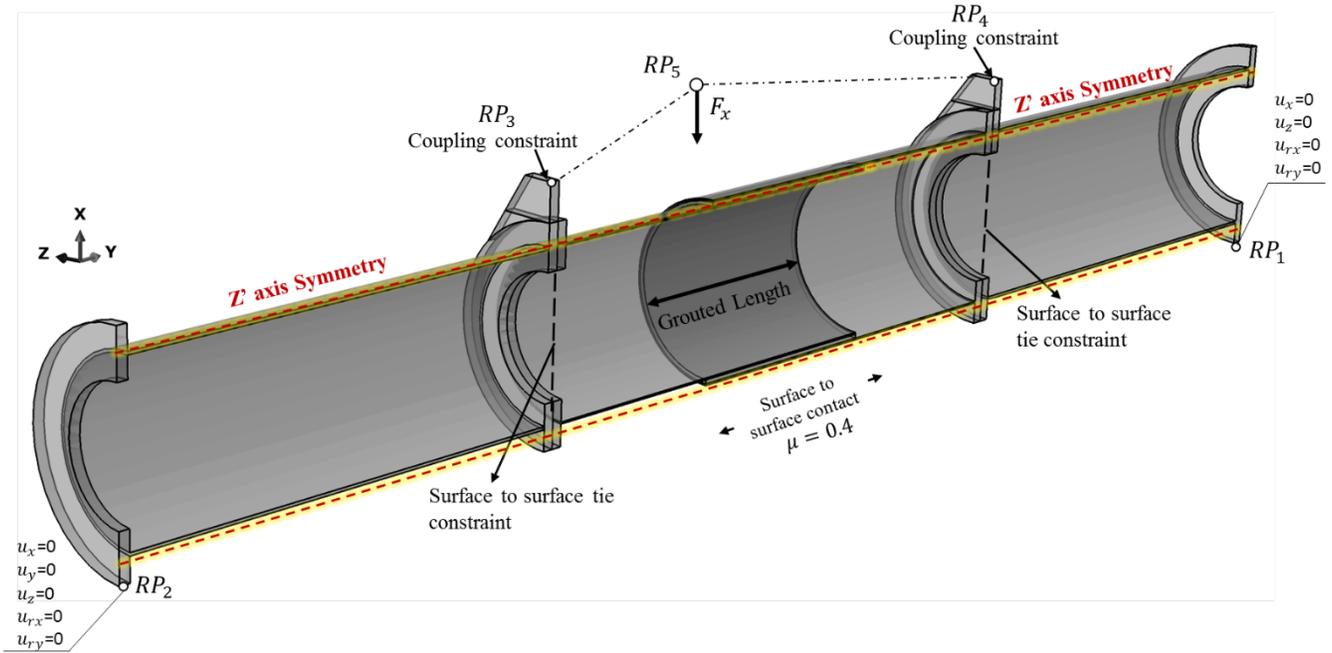
their role was simulated using appropriate constraints and boundary conditions reflecting the exact set-

171

up presented in [7]. A detailed illustration of the selected constraints and applied boundary conditions

172

used in the FE models is shown in figure 4.



**Figure 4.** FE model description, boundary conditions and constraints

173

174

175 Initially, the symmetry of the assembly was exploited by modelling half of the specimen and applying a  
 176 symmetry boundary condition towards z-axis. To apply the boundary conditions, reference points were  
 177 introduced on the flanges. The node-sets of each flange are tied with kinematic coupling constraints to  
 178 the corresponding reference points. The simply-supported boundary conditions were applied to reference  
 179 points  $RP_{1,2}$  as shown in figure 4, whereas the load was applied to reference points  $RP_{3,4}$  with an  
 180 eccentricity of 350 mm using a smooth amplitude function. The models were subjected to a load of 1000  
 181 kN and then unloaded to imitate the experimental campaign's loading scheme. Within this loading  
 182 protocol, grout cracking, separation and yielding of the pile occurred. The end-beams were tied to the  
 183 pile and sleeve with tie constraints to represent the bolted flanges that were used in test. The interactions  
 184 between the grout, sleeve and pile were resolved with a surface to surface scheme. Hard contact was set  
 185 in the normal direction, which allows for compressive stresses to be transferred at the interfaces and  
 186 enables gap development between steel and grout. In the tangential direction a penalty contact  
 187 formulation with a coefficient of friction of  $\mu = 0.4$  was selected.

### 188 **3.3 Material modelling**

#### 189 **3.3.1 Steel**

190 The steel parts of the specimen were described as Elastic-Plastic with an isotropic hardening behaviour.

191 The behaviour of the S235 steel tubes was defined according to the tensile coupon tests [7]. The true

192 stress,  $\sigma_{true}$ , and strain,  $\varepsilon_{true}$ , curve was obtained from the engineering stress,  $\sigma_{eng}$ , and strain,  $\varepsilon_{eng}$ ,

193 following equations (1) and (2):

$$194 \quad \sigma_{true} = \sigma_{eng}(1 + \varepsilon_{eng}) \quad (1)$$

$$195 \quad \varepsilon_{true} = \ln(1 + \varepsilon_{eng}) - \left(\frac{\sigma_{true}}{E}\right) \quad (2)$$

196 The Young's modulus ( $E$ ) and Poisson's ratio ( $\nu$ ) was set to 210 *GPa* and 0.3, respectively. A density of

197 7850  $kg/m^3$  was used for steel.

#### 198 **3.3.2 High strength grout**

199 A HSG was the employed cementitious medium with a mean compressive strength ( $f_{cm}$ ) of 130 *MPa*.

200 The material response of HSGs has similar characteristics to high and ultra-high strength concrete. In

201 this direction, the CDP model [20, 21] is chosen in ABAQUS, which allows the definition of the grout

202 behaviour in compression and tension and can trace crushing and cracking.

203 For the compressive behaviour of the material and to consider the confinement to some extent, the

204 ascending branch was assumed to be linear until the peak strength is reached. In normal concrete, the

205 peak strain ( $\varepsilon_{cl}$ ) is often assumed to be equal to 0.0022, however this approach is limited to concrete

206 with a compressive strength of up to 80 *MPa* [22]. Thereby, the peak strain was adjusted based on the

207 analytical formulation proposed in [23] to better-reflect the HSG properties, which reads as:

208 
$$\varepsilon_{c1} = \frac{0.7f_{cm}^{0.31}}{1000} \quad (3)$$

209 Following the peak strain, the descending branch was set according to model-code CEB-FIP [22]. The  
210 damage variable  $d_c$  ranged from 0 to 1 and was determined according to [18]:

211 
$$d_c = 1 - \left(\frac{f_{cm}}{\sigma_c}\right) \quad (4)$$

212 where  $\sigma_c$  is the stress corresponding to the inelastic strain.

213 To define the tensile behaviour of the grout the fracture energy approach [24] was adopted, as the strain  
214 formulation has been reported to cause numerical instabilities in concrete-related studies [25]. The  
215 fracture energy was determined from [22] as follows:

216 
$$G_F = G_{F0} - \left(\frac{f_{cm}}{f_{cm0}}\right)^{0.7} \quad (5)$$

217 where  $G_{F0}$  is the base value for fracture energy as a function of the aggregate size and  $f_{cm0} = 10 \text{ MPa}$ .

218 Damage in tension was defined similarly to compression. The remaining properties of the HSG and the  
219 CDP parameters are summarised in Table 2. The global response of the model was found to be more  
220 sensitive to dilation angle ( $\psi$ ) values, hence the selected value for the parameter must be based on the  
221 grout material used and should always be calibrated against experimental data. In this study, a value of  $\psi$   
222 =  $38^\circ$  was found to result in an appropriate global response when compared to test results.

223

**Table 2:** HSG properties and CDP parameters

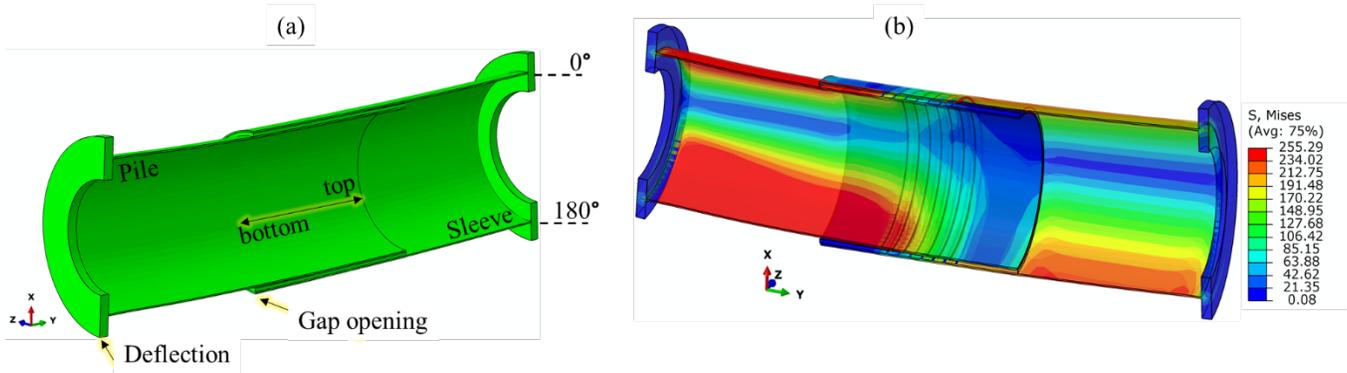
Description/Symbol	Value/Unit	CDP	Value
Modulus of Elasticity, $E$	50000 [MPa]	Dilation angle, $\psi$	38°
Poisson's ratio, $\nu$	0.19	Eccentricity	0.1
Density, $\rho$	2380 [kg/m <sup>3</sup> ]	Compressive yield stress Uniaxial yield stress	1.162
Tensile strength, $f_t$	7 [MPa]	Viscosity	0
Fracture Energy, $G_F$	150.8 [Nm/m <sup>2</sup> ]	$K_c$	2/3

#### 225 **4. Model validation**

226 The global behaviour of the FE models was validated by comparing load-deflection curves, stresses on  
 227 the steel piles and interface opening. The notation which will be followed in the remaining sections is  
 228 presented in figure 5a along with the locations where deflection and gap opening is measured unless  
 229 otherwise stated. The pile and sleeve top correspond to 0 degrees, whereas the bottom to 180 degrees.  
 230 The gap opening will always refer to the bottom of the GC at a location of 180 degrees. On both GCs  
 231 pile yielding occurs at the bottom of the GC at 0 degrees (see figure 5b). Both models exhibited the  
 232 expected interface de-bonding in the opposing top and bottom end, however in the SKGC model this  
 233 was significantly reduced. The interface gap at the top and bottom of the connection was an expected  
 234 outcome and was also documented in the test results.

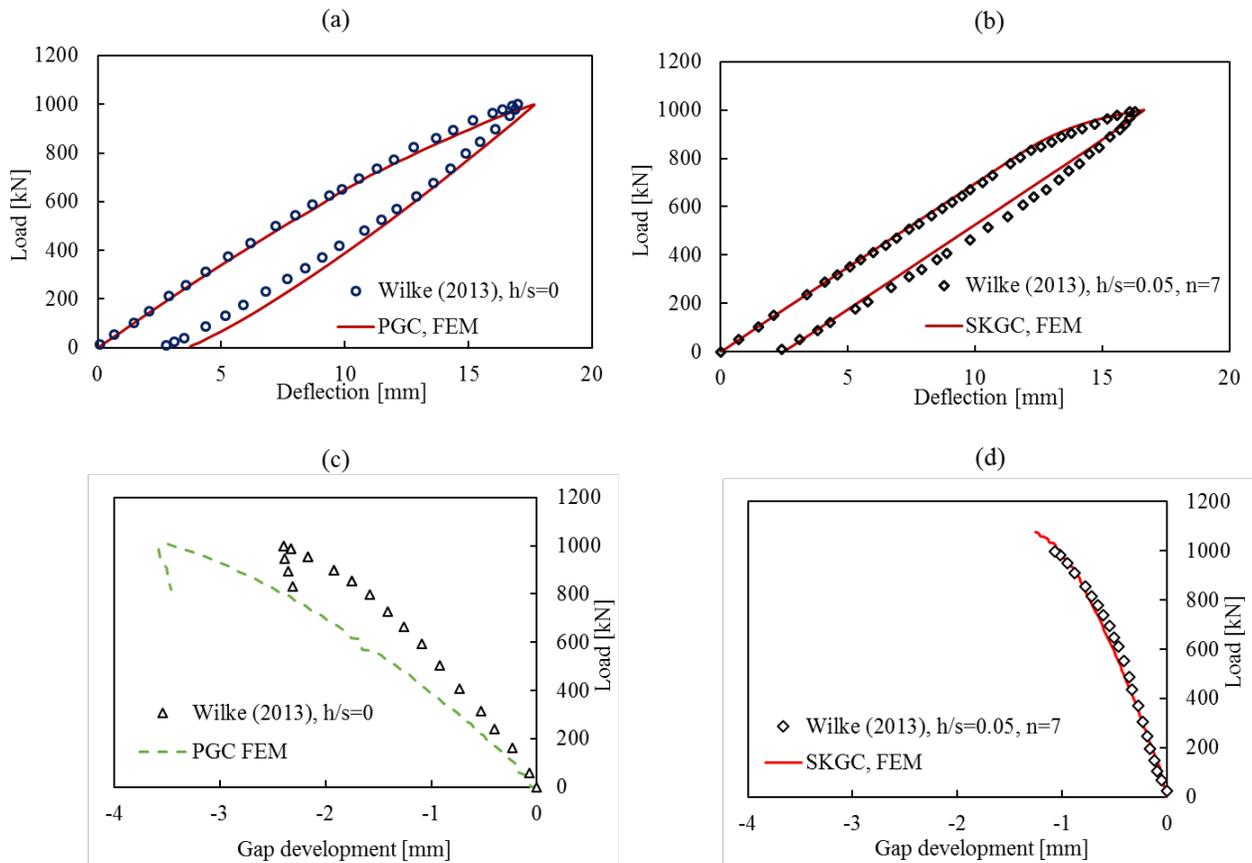
235 The load-deflection curves for PGC and SKGC are plotted in figure 6a and 6b and the gap development  
 236 from both models is depicted in figures 6c and 6d. Overall the FE models are in very good agreement  
 237 with the experimental data and their ability to replicate the response of the specimens numerically is

238 demonstrated. Finally, when comparing the peak displacement of the two models, a 6% higher  
 239 deflection was observed for the PGC, which is almost identical to the one monitored during the tests.



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 241  
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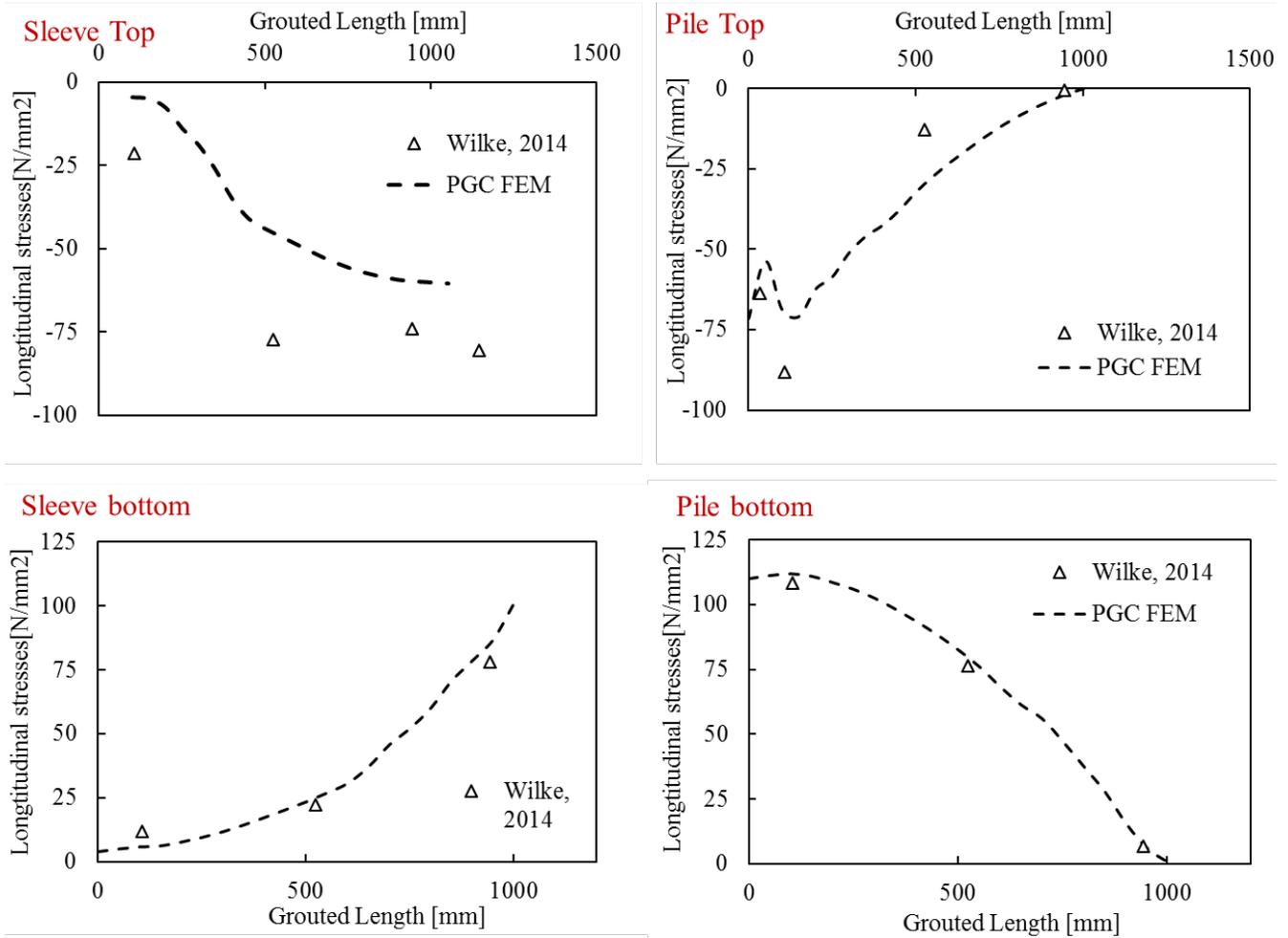
**Figure 5.** a) GC notation used for the FE models and b) Mises stresses on GC



243  
 244  
 245

**Figure 6.** Load-deflection comparison between FE models and experimental data. a) PGC, b) SKGC. Load-gap opening curve for c) PGC and d) SKGC.

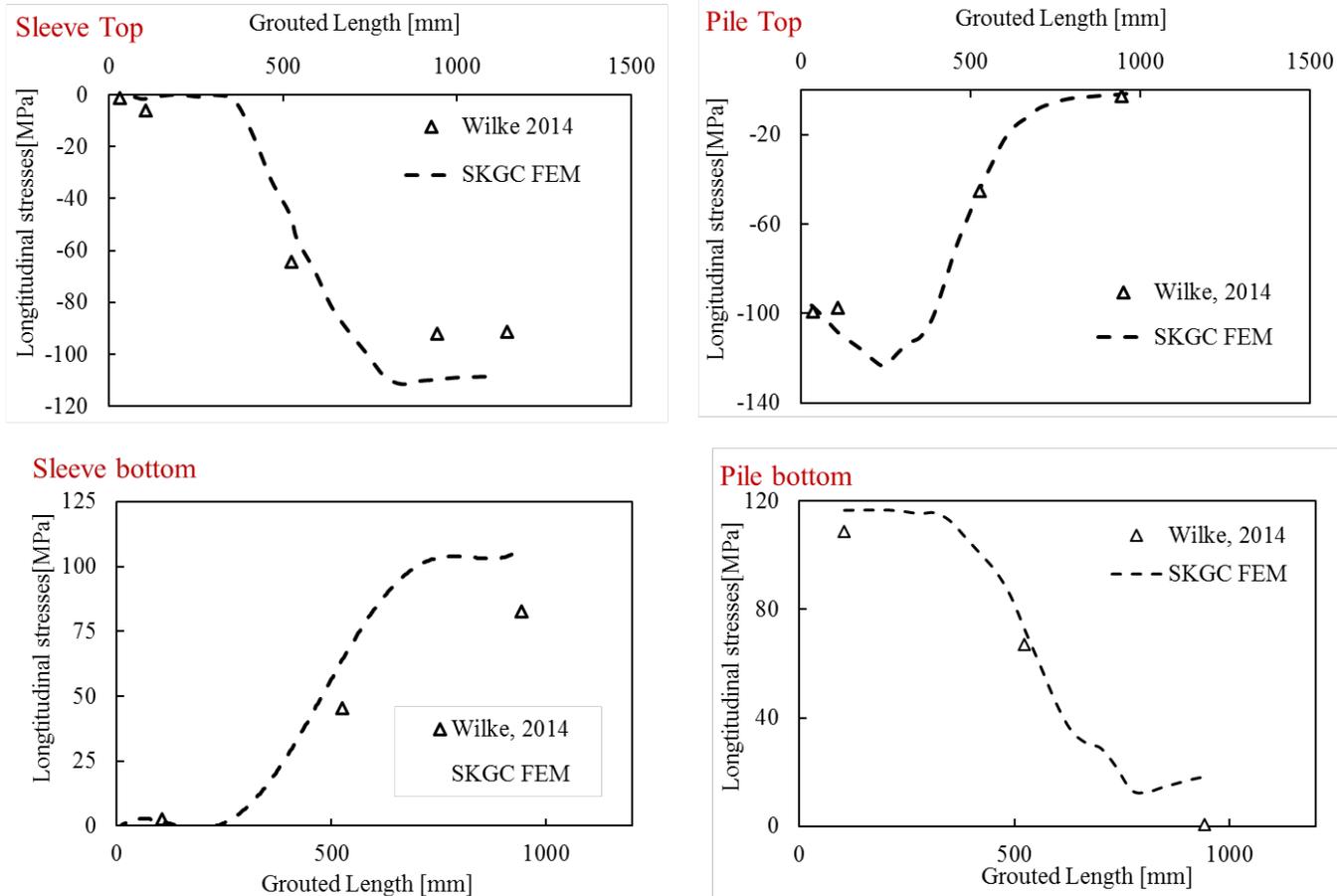
246 To enhance the validity of the FE models, the longitudinal stresses ( $\sigma_{22}$ ) towards the length of the  
 247 connection are also illustrated in figures 7 and 8. For both models, good agreement is achieved and only  
 248 the stresses at the top of the PGC sleeve are slightly underestimated.



249

250

**Figure 7.** PGC longitudinal stresses at F = 435 kN



251  
252 **Figure 8.** SKGC longitudinal stresses at F =435 kN

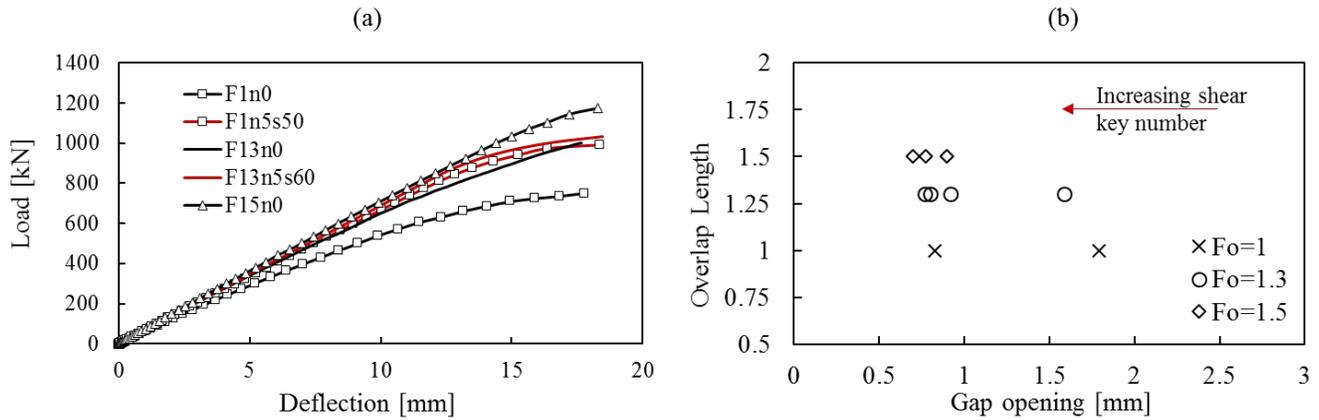
253 **5. Parametric analysis**

254 The verified FE models were taken forward to carry out a parametric study and provide further insight  
 255 on the effect of geometrical parameters on the behaviour of GCs. The focus of the analyses was set on  
 256 shear key parameters on GCs with various overlap lengths. More specifically, the number of shear keys  
 257 ( $n$ ) on each pile and the spacing ( $s$ ) between them were examined. The main point of interest was  
 258 initially the global behaviour of the model – involving stiffness and interface openings, and finally the  
 259 local behaviour, focusing on crack patterns and failure modes. For illustration purposes two plain  
 260 connections were also solved to highlight the interlock effect on the connection provided from the  
 261 inclusion of shear keys.

262 A detailed description of the model geometries used in this study is given in Table 3. The tabulated  
263 dimensions refer to parts of the model that form the connection while the remaining steel tubes and ring-  
264 flanges were maintained as reported in Table 1. All investigated models were subjected to the same  
265 constraints, material properties and boundary conditions described in sections 2.1 and 2.2 to comply  
266 with the validation study and the same failure criterion was followed. The notation of the models is  
267  $F_{o,i} - n_i - s_i$ , where  $F_{o,i}$  refers to overlap length and  $n, s$  to the corresponding number of shear keys and  
268 spacing, respectively. For instance,  $F_o15n7s50$  refers to a GC with  $F_o = 1.5$  and 7 shear keys equally-  
269 spaced every 50 mm. For all the subsequent parametric models an effective number of shear keys ( $n_{eff}$   
270  $=n+1$ ) has been used on the sleeve.

## 271 **5.1 Shear key number**

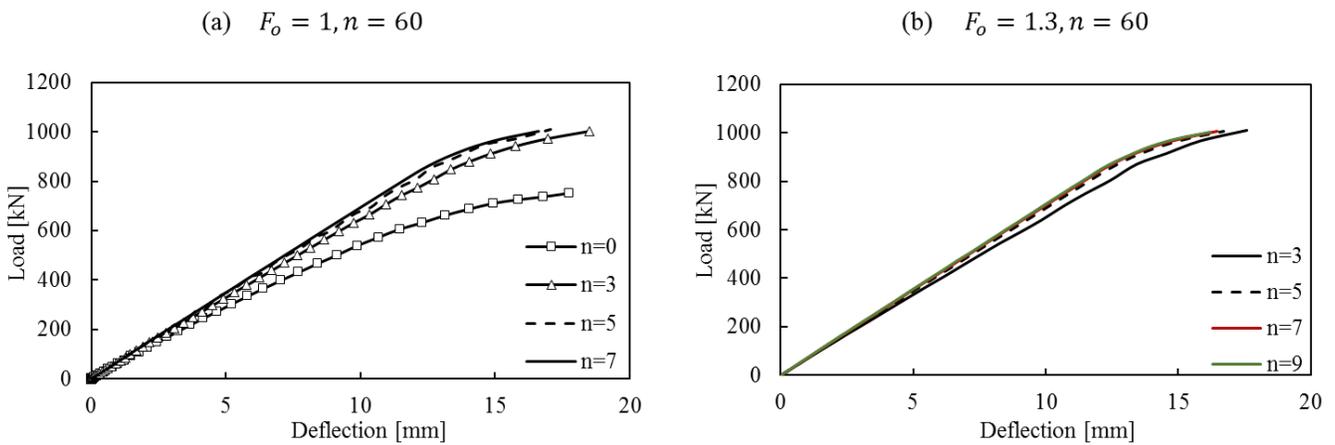
272 Experimental campaigns conducted to date, often involve connections with  $F_o < 1.5$ , although this lies  
273 outside the recommended limits by DNV [9]. Initially, to demonstrate the benefit in bending stiffness  
274 with increasing grouted lengths, a typical force-displacement curve of selected models is shown in  
275 figure 9a. Models with shear keys are also included to demonstrate the superior performance exhibited  
276 when compared to plain connections. A series of numerical models with varying shear keys were  
277 developed to monitor their effect on the connection. In agreement with the validation study, the interface  
278 gap occurring at the opposing sides of the grout is minimised, with an increase in the number of shear  
279 keys (figure 9b), illustrating the considerable effect shear keys and grouted length have on the overall  
280 performance.



281

282 **Figure 9.** a) Force–displacement curve illustrating the effect of  $F_o$ , b) Maximum interface opening at  
 283 GC bottom

284 In figure 10, a comparison of the global response of representative models dictates the benefits from  
 285 increasing the number of installed shear keys. This is particularly pronounced for the lowest overlap  
 286 length. From the parametric analysis it was observed that the influence of the shear key number is  
 287 reduced for higher overlap lengths ( $F_o=1.5$ ).



288

289 **Figure 10.** Force-deflection curves for: **a)**  $F_o=1$ , **b)**  $F_o=1.3$

290 The gap that develops in the steel grout interfaces due to bending is of considerable interest, as it leads  
 291 to water ingress. This is because it results in reduced friction between steel and grout, which can

292 effectively disrupt the performance of the joint [26]. According to recent studies [27], GCs tested in dry  
 293 environment have superior performance when compared to GCs in wet conditions.

294 To investigate the interface behaviour of the numerical models the maximum gap opening at the bottom  
 295 of the connection was determined for the maximum applied load. Thereinafter, the results are compared  
 296 with the analytical model proposed in [5] which is described in equations 6 to 8.

$$297 \quad p_{nom} = \frac{3 \pi M_{tot} E L_g}{E L_g [R_p L_g^2 (\pi + 3\mu) + 3\pi \mu R_p^2 L_g] + 18\pi^2 k_{eff} R_p^3 \left[ \frac{R_p^2}{t_p} + \frac{R_{TP}^2}{t_{TP}} \right]} \quad (6)$$

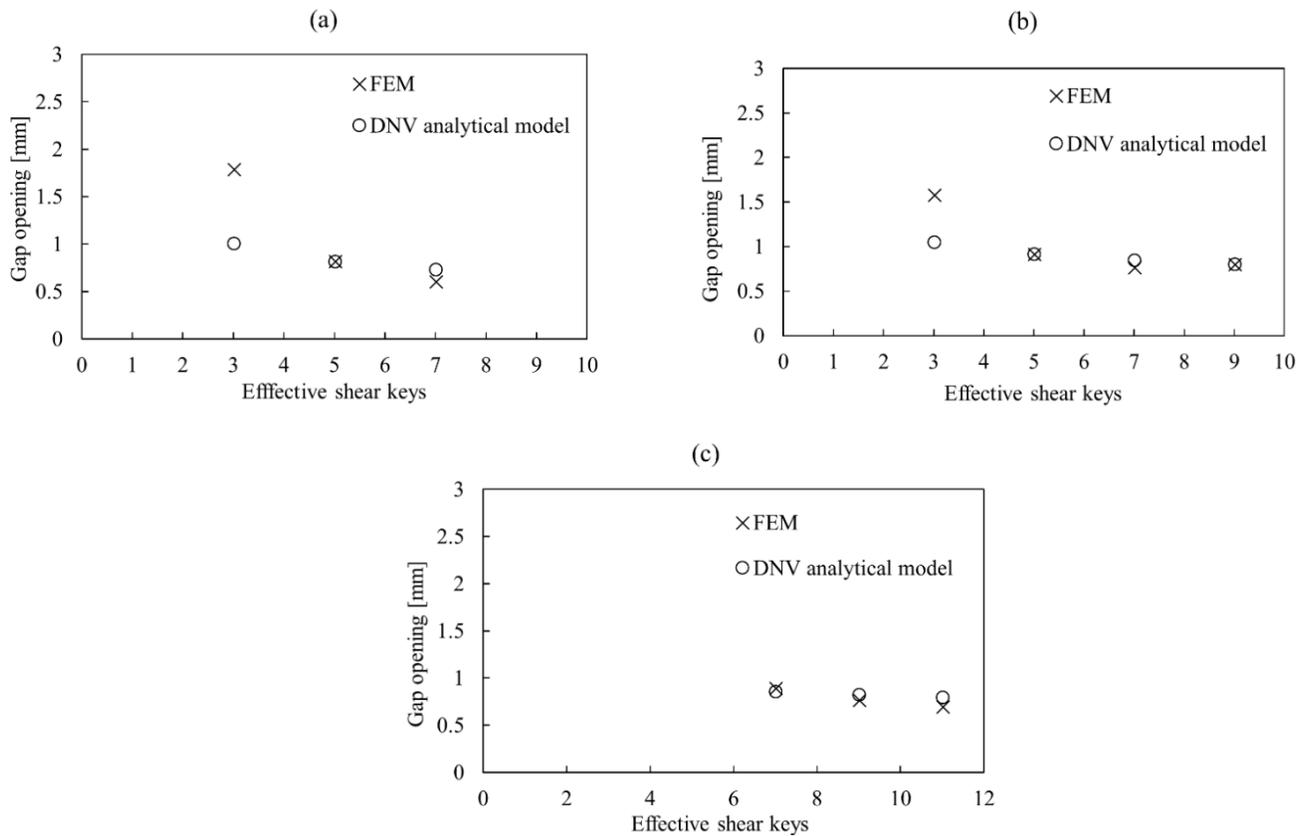
298 where  $p_{nom}$  is the nominal contact pressure,  $M_{tot}$  the total moment and  $k_{eff}$  the effective spring stiffness  
 299 which reads:

$$300 \quad k_{eff} = \frac{2 t_{TP} s_{eff}^2 n E}{4 \sqrt[4]{3(1-\nu^2)} t_g^2 \left[ \left( \frac{R_p}{t_p} \right)^{3/2} + \left( \frac{R_{TP}}{t_{TP}} \right)^{3/2} \right] t_{TP} + n s_{eff}^2 L_g} \quad (7)$$

301 where  $s_{eff}$  is the effective spacing of shear keys reduced by one shear key width. Thus, the vertical  
 302 opening can be calculated as follows:

$$303 \quad \delta_u = \frac{6 p R_p}{E L_g} \left( \frac{R_p^2}{t_p} + \frac{R_{TP}^2}{t_{TP}} \right) \quad (8)$$

304 In figure 11, the opening that developed on GCs with  $h/s = 0.05$  is compared with the prediction from  
 305 the analytical model. For most of the investigated geometries very good agreement was found. The  
 306 biggest discrepancies between the numerical and analytical solutions appeared for the models with lower  
 307 shear key numbers. It is apparent that an increased number of shear keys minimises the gap in the  
 308 interface for all the examined grouted lengths.



309

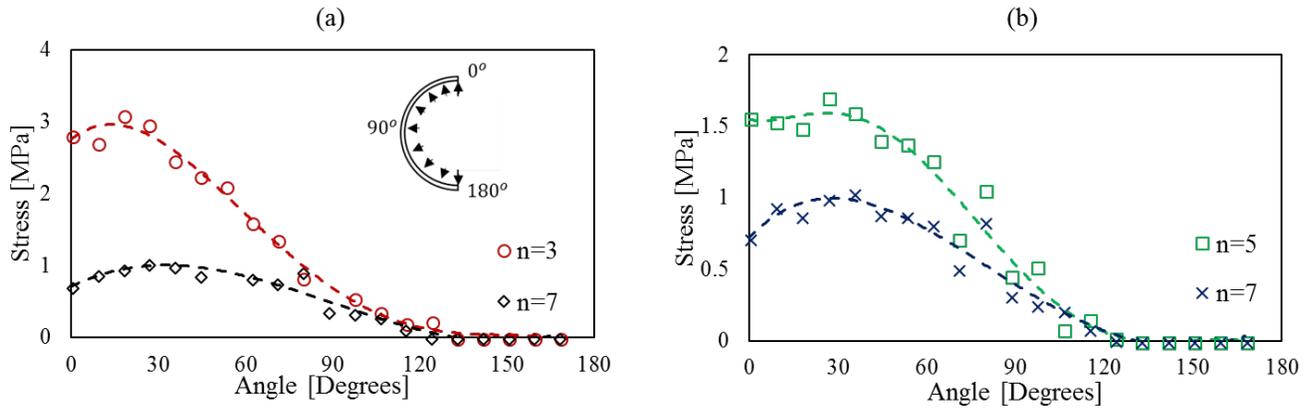
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**Figure 11.** Maximum developed gap at  $M_{max}$  for **a)**  $F_o=1$ , **b)**  $F_o=1.3$ , **c)**  $F_o=1.5$

311

The use of shear keys alters the force transfer mechanism compared to a plain connection. The bending moments are mainly-transferred through the shear key region, which subsequently reduces the stresses at the top and bottom of the connection. This is illustrated in figures 12a and 12b where the pressure along the circumference of the sleeve is plotted for models with varying shear keys and overlap lengths. The stresses shown, are exerted on the inner sleeve surface, at the bottom of the GC following the notation of figure 5a. Three neighbouring circular paths on the sleeve were used to extract the stresses from the sleeve nodes. The data points shown in figure 12 depict the average values of the stresses from the three nodes on each location of the circumference. A fourth order polynomial is fitted to the data-sets to highlight the distribution of stresses along the circumference of the sleeve.

319

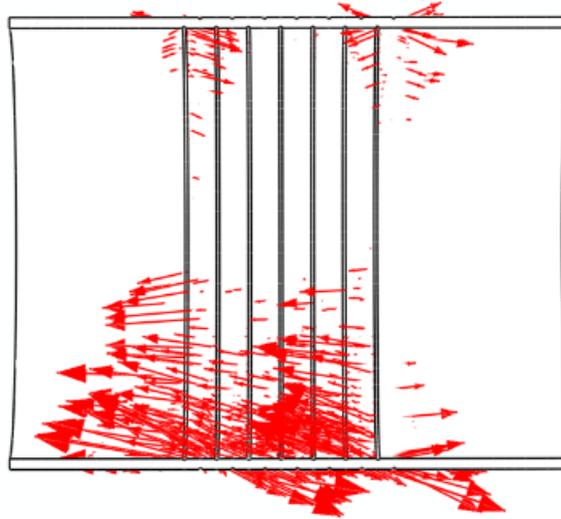


320

321 **Figure 12. a)** Contact pressure at maximum opening around the sleeve circumference for **a)**  $F_o=1$ , and  
 322 **b)**  $F_o=1.3$

323 Considering the results from the presented FE models an increase in the effective number of shear keys  
 324 is shown to be beneficial as contact pressure reduces significantly at the measured location confirming  
 325 that the stresses at the GC ends are reduced when using additional shear keys. Connections with a higher  
 326 number of shear keys exhibit lower stresses, particularly in the region between  $0^\circ$  and  $90^\circ$  where contact  
 327 pressure peaks. The stresses on the sleeve reduce to zero when approaching  $180^\circ$  as opening has  
 328 occurred at the tensile side of the tube. The findings are in agreement with previous studies [2, 7], as the  
 329 shear-key region is now transferring a higher proportion of the applied loads when compared to a plain  
 330 GC.

331 The alteration in load-transfer can also be realised if one considers the intensity of plastic strains in the  
 332 shear key vicinity. In figure 13 the direction of plastic strains on a grout core are shown. The increased  
 333 population of arrows in the tensile shear-keyed region depicts the damage occurrence taking place in this  
 334 area.



335

336

**Figure 13.** Grout plastic strain vectors at maximum load level

337

## 5.2 Grout failure modes

338

Previous experimental campaigns [7, 28] have noted that grout failure modes depend on shear key

339

spacing. Due to spacing, cracking patterns within the grout core vary, from diagonal cracks with

340

different inclinations between opposing shear keys, to cylindrical failure surfaces initiating at the tip of

341

the shear keys. In all cases the grout failure in the shear key region typically develops along the

342

circumference. Within this parametric study the ability of the FE model to capture the alternative failure

343

modes of the grout based on shear key spacing has been examined. For this purpose, three representative

344

FE models with varying shear key spacing ( $s = 30, 60, 120$  mm) and a fixed overlap length ( $F_o = 1.3$ )

345

were solved. The shear key height ( $h = 3$  mm) and the grout compressive strength (130 MPa) were kept

346

constant for all GCs to isolate the effect of spacing. The selected distances resulted in the following

347

ratios:  $h/s = 0.025, 0.05, 0.1$ . In this study the CDP model was used, therefore cracking can be effectively

348

traced by means of plastic strains.

349

In figure 14 iso-surface and banded contour plots of the grout plastic strains are illustrated. The location

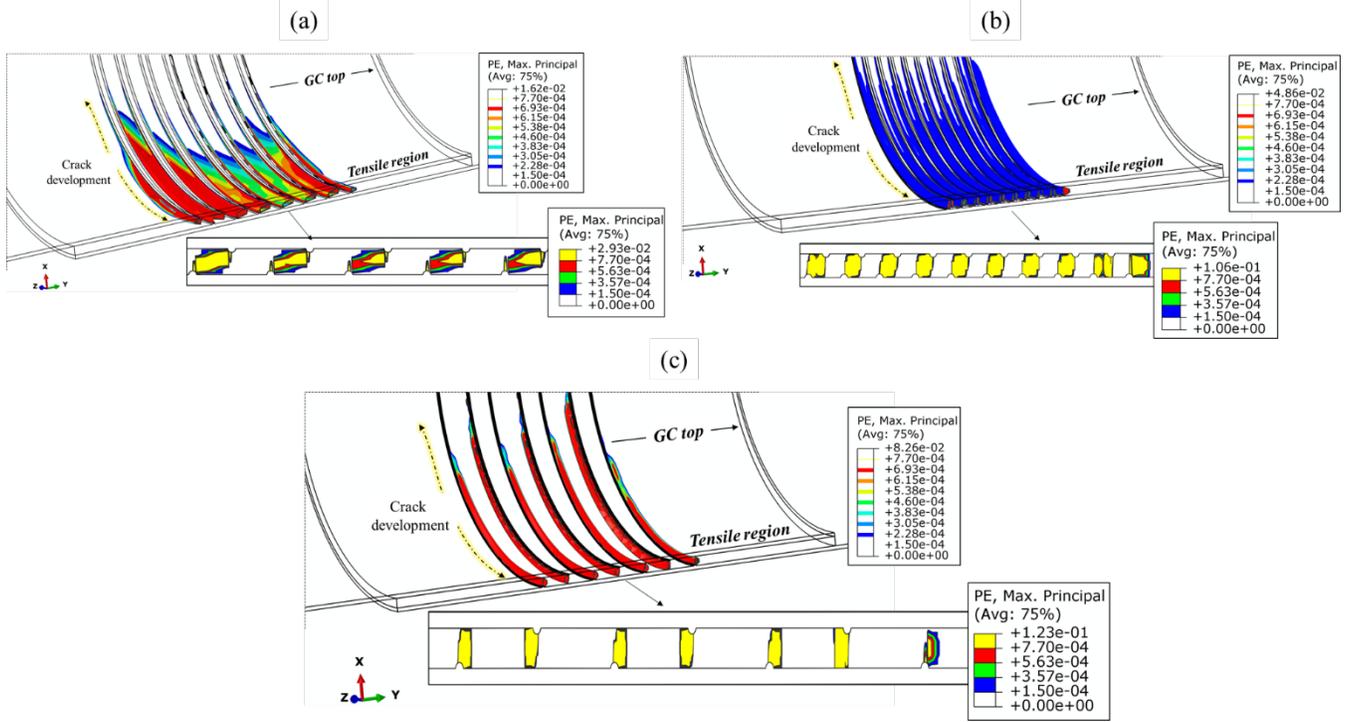
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of interest is set on the tensile region of each GC focusing on the strut development based on the shear

351 key positioning. For all the arrangements, cracking initiated at the tip of the shear keys as expected. For  
352 a spacing of 60 mm (figure 14a), cracks developed from pile and sleeve shear keys until they merge to  
353 form a diagonal strut which enables the load-transfer mechanism. Once the load increases additional  
354 wedged cracks form in front of the shear key. The inclination of the strut between two shear keys for this  
355 configuration was found to be  $34^\circ$  (figure 15a). This cracking behaviour compares well with findings  
356 reported on previous experimental studies involving GCs with similar shear key spacing and ratios [29].  
357 Once the spacing of the shear keys was reduced to  $s=30$  mm, the diagonal strut formed appeared with a  
358 steeper inclination of  $53^\circ$  as illustrated in figure 15b. Despite the high  $h/s$  ratio the strut inclination was  
359 found to be within acceptable limits.

360 For  $s=120$  mm the contour plots are shown in figures 14c, 15c. The increasing distance between the  
361 shear key leads to a very low shear connector ratio aiming to provoke the change in failure mode.  
362 Initially, due to the arrangement, the cracks initiated in front of the pile shear keys in contrast to the  
363 previous models. As loading increases cracks originate from the shear key tip until they reach the  
364 opposing pile or sleeve surface. Once failure of the strut has occurred, a cylindrically-shaped failure  
365 surface initiates circumferentially for all the shear keys. The same mechanism was reported for all shear  
366 keys of the GC. The angle of the diagonal cracking was calculated for all FE models based on ideal  
367 shear key distances and dimensions without considering welding irregularities.

368 With this section the robustness and ability of the numerical model to predict changing failure modes  
369 which are caused by shear key spacing was demonstrated convincingly. Thus, the presented numerical  
370 scheme can be used as a solid foundation for the design of GCs based on FEA.

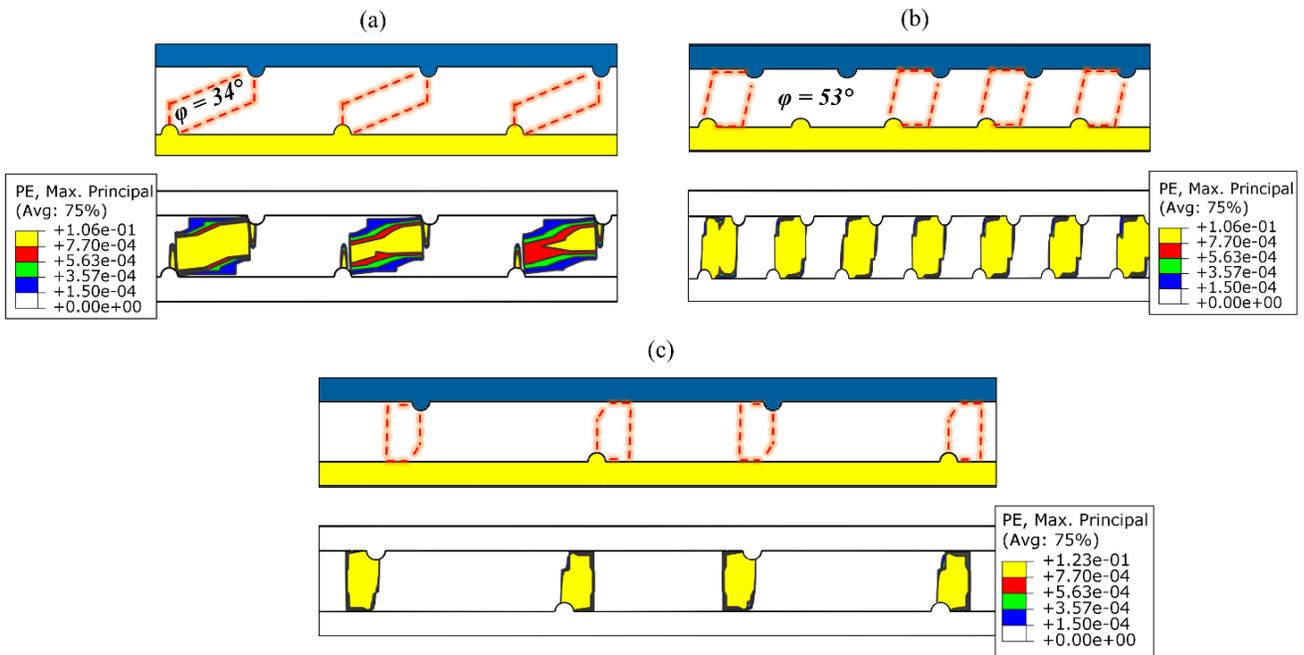


371

372 **Figure 14.** Iso-surface and contour plots of grout plastic strains for a)  $s=60$  mm, b)  $s=30$  mm, c)  $s=120$

373

mm



374

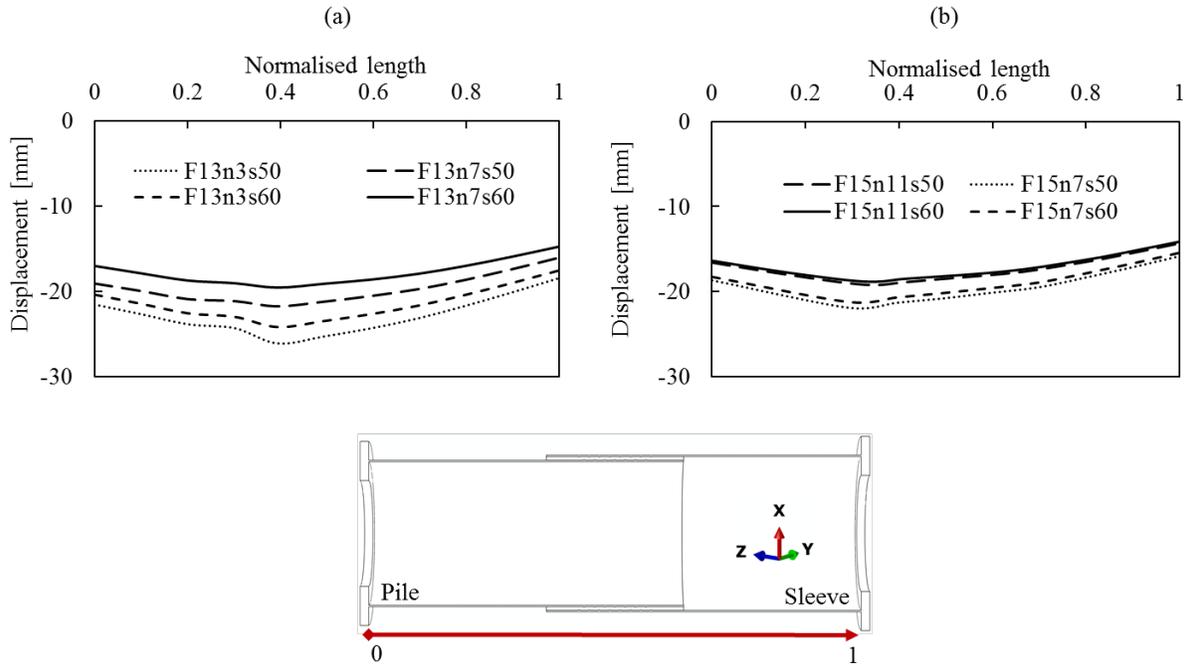
375 **Figure 15.** Grout failure modes for varying shear key spacing. a)  $s=60$  mm, b)  $s=30$  mm, c)  $s=120$  mm

### 376 **5.3 Shear key ratio**

377 The upper limit suggested in design guidelines [9] for GCs with shear connectors is set to  $h/s < 0.1$ ,  
378 however, previous studies involving connections under bending have mostly employed lower shear key  
379 ratios from 0.02 to 0.05 [7, 28, 29]. Similar ratios can be found in the literature for axially loaded GCs.  
380 In order to study the influence of higher ratios, nine models with a ratio of 0.06 were numerically  
381 solved. To achieve this ratio the spacing between two consecutive shear keys was reduced to 50 mm  
382 maintaining the same height and width for the shear keys.

383 In figures 16a, b the maximum displacement is depicted for models of different overlap lengths. For  
384 comparison purposes, the location axis has been normalised against the total length between the pile and  
385 sleeve flange as shown figure 16. Therefore, a location equal to zero, corresponds to the displacement of  
386 the pile flange and a location equal to 1 corresponds to the sleeve flange. For all the analysed models  
387 with various  $F_o$  a consistent pattern is noticed. Although a stiffer response is depicted for an increased  
388 overlap length and shear key number as discussed in section 4.2, increasing the shear key ratio did not  
389 improve the performance of the connection. The lower growth in displacement for GCs with  $h/s=0.05$ ,  
390 resulted in a better bond action between the grout and steel. Likewise, with the results in section 4.1, for  
391  $h/s=0.06$  the influence of the parameter is noticeable for lower  $F_o$ , but significantly declines for  $F_o = 1.5$   
392 (figure 16b).

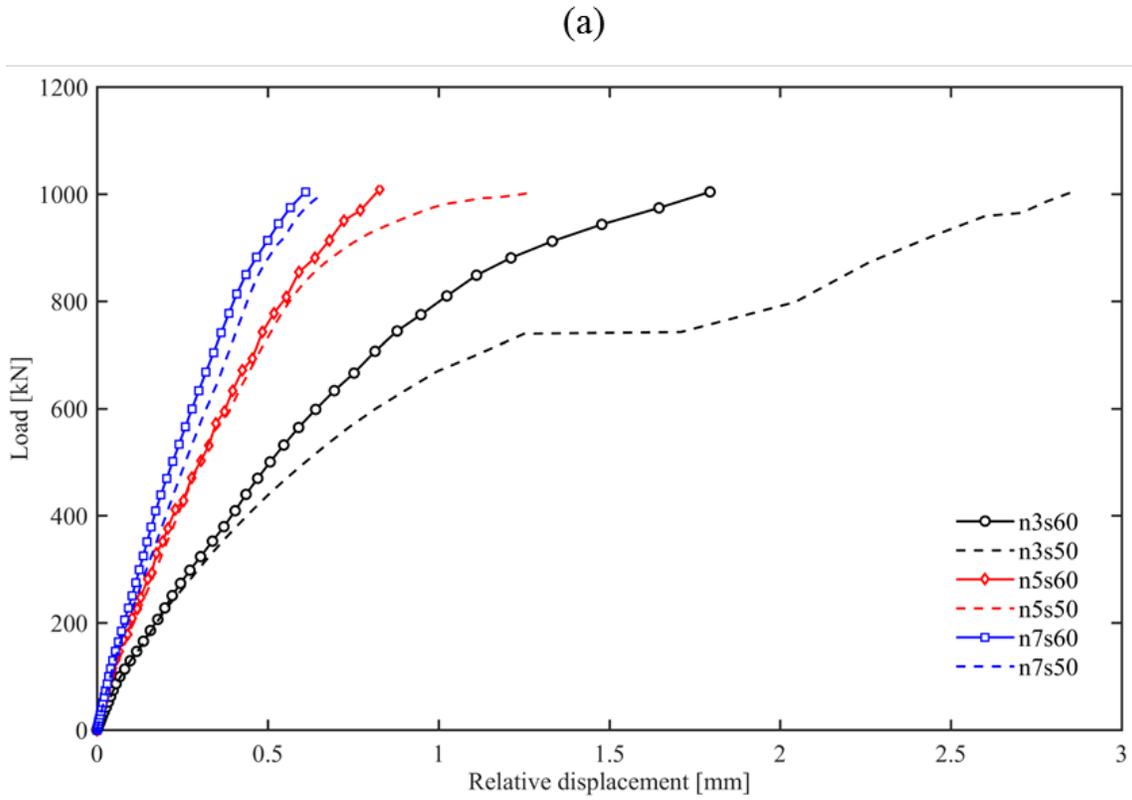
393 In addition, as depicted in figure 17 the relative displacement which was found between the pile and  
394 sleeve for the parametric models is significantly higher for models with a shorter shear key region.  
395 Another deduction from figure 17 is that gaps on the interface can take place at lower load levels.  
396 Particularly for models with shorter shear key regions (see, e.g.,  $F_o1n3s50$  and  $F_o15n3s60$ ) significant  
397 gaps occurred rapidly at  $F \sim 0.6F_{max}$ .



398

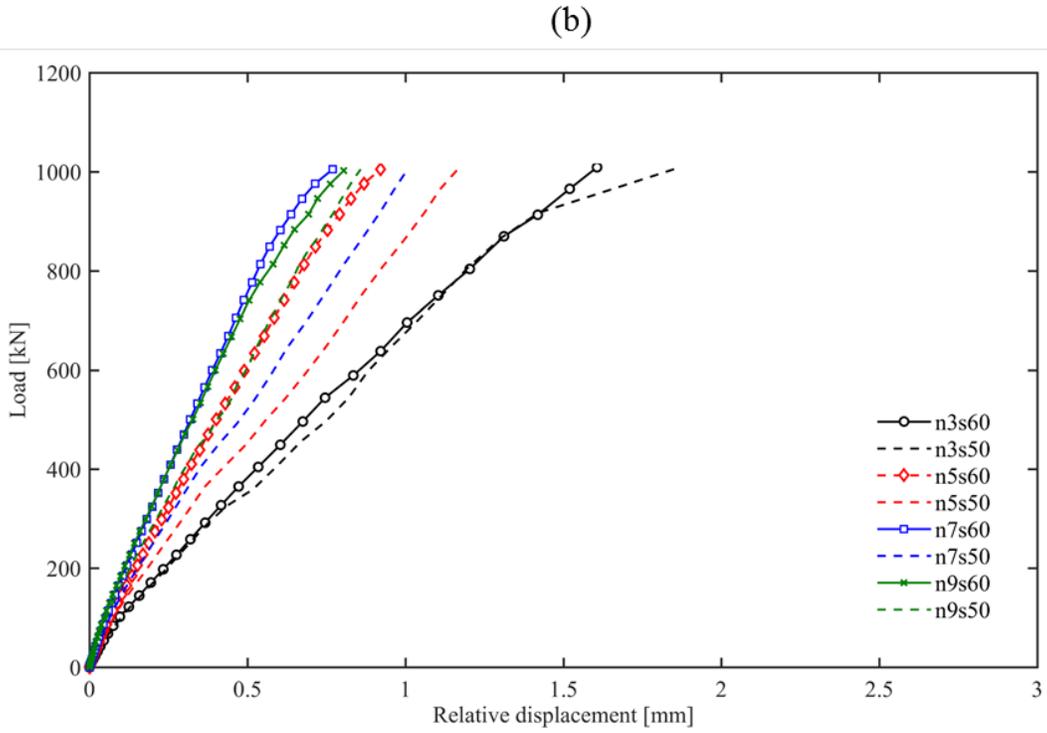
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**Figure 16.** Displacement growth over normalised length for: **a)**  $F_o=1.3$ , **b)**  $F_o=1.5$

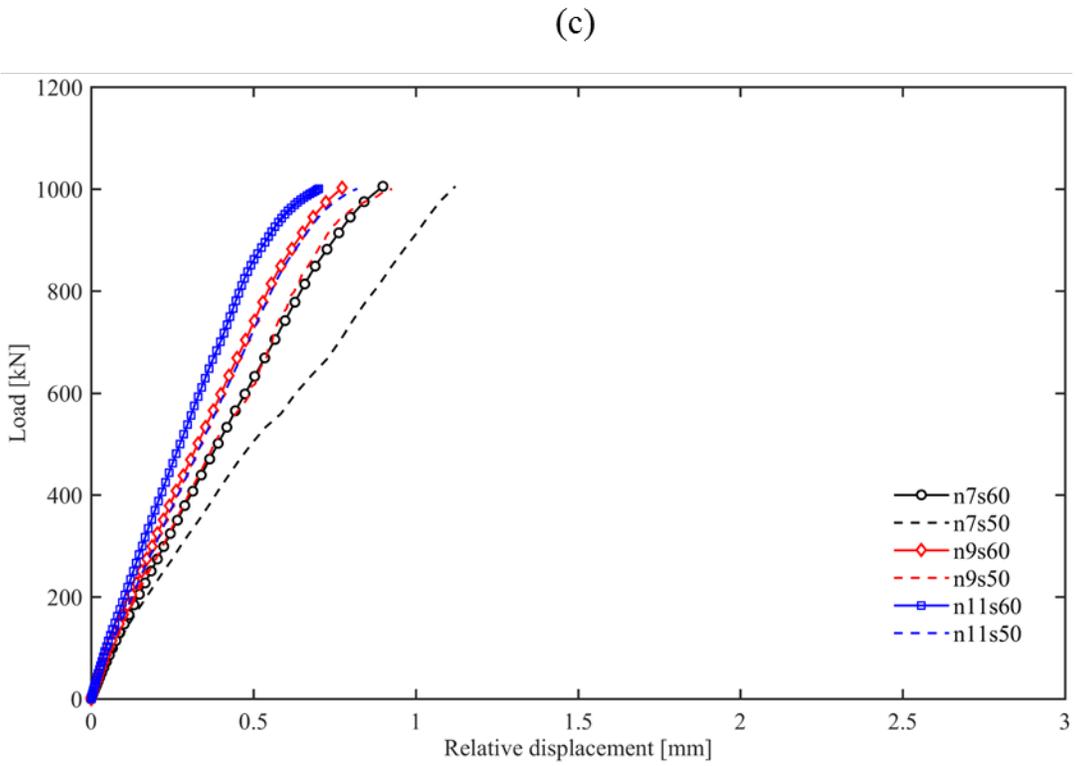


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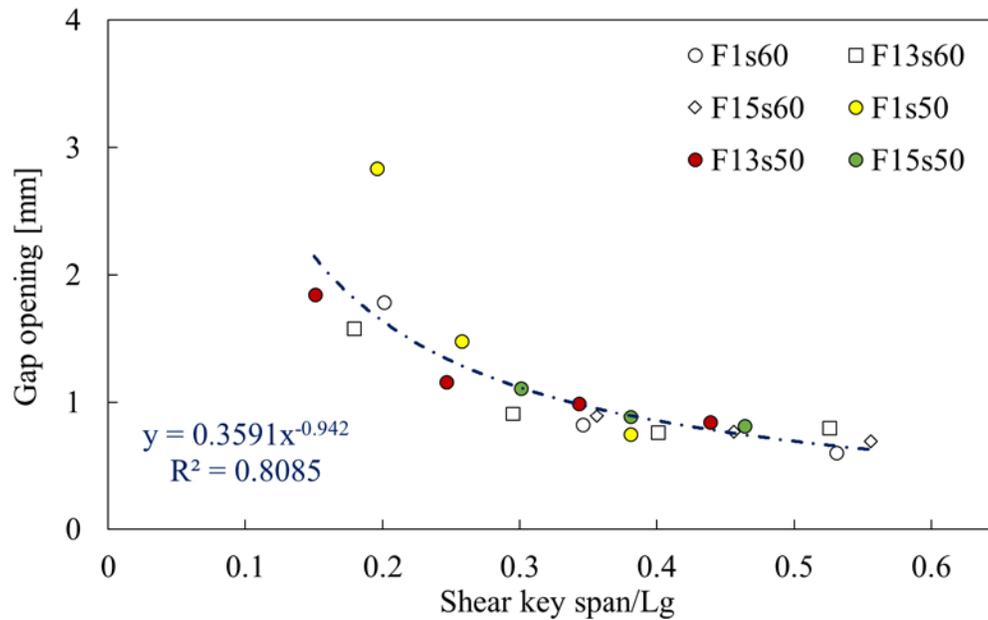
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403

**Figure 17.** Force–Relative displacement for a)  $F_0=1$ , b)  $F_0=1.3$ , c)  $F_0=1.5$

404 Finally, in figure 18 an overview of the interface gap calculated from the parametric models is presented  
 405 as a function of the shear key region of each GC. The developed gaps form a plateau when the shear key  
 406 regions are close to half the grouted length. However, it is evident that connections with shear keys in  
 407 the middle third of the grouted length or less would benefit from a higher number of shear keys. An  
 408 increasing number of shear keys is significantly reducing the de-bonding of the connection particularly  
 409 for low overlap lengths.



410

411

**Figure 18.** Maximum gap opening against shear key span from parametric models

412

**Table 3.** Geometrical characteristics of FE models for parametric analysis

Name	$F_o$	$n$	Dimensions [mm]				
			$L_{p,s}$	$t_{s,p}$	$h$	$s$	$L_g$
$F_o1n0s0$	1	-	1835	8	n/a	n/a	800
$F_o1n3s60$	1	3	1835	8	3	60	800
$F_o1n3s50$	1	3	1835	8	3	50	800
$F_o1n5s60$	1	5	1835	8	3	60	800
$F_o1n5s50$	1	5	1835	8	3	50	800
$F_o1n7s60$	1	7	1835	8	3	60	800
$F_o1n7s50$	1	7	1835	8	3	50	800
$F_o13n0s0$	1.3	-	1955	8	n/a	n/a	1040
$F_o13n3s60$	1.3	3	1955	8	3	60	1040
$F_o13n3s50$	1.3	3	1955	8	3	50	1040
$F_o13n3s120$	1.3	3	1955	8	3	120	1040
$F_o13n9s30$	1.3	9	1955	8	3	30	1040
$F_o13n5s60$	1.3	5	1955	8	3	60	1040
$F_o13n5s50$	1.3	5	1955	8	3	50	1040
$F_o13n7s60$	1.3	7	1955	8	3	60	1040
$F_o13n7s50$	1.3	7	1955	8	3	50	1040
$F_o13n9s60$	1.3	9	1955	8	3	60	1040
$F_o13n9s50$	1.3	9	1955	8	3	50	1040
$F_o15n0s0$	1.5	-	2035	8	n/a	n/a	1200
$F_o15n7s60$	1.5	7	2035	8	3	60	1200
$F_o15n7s50$	1.5	7	2035	8	3	50	1200
$F_o15n9s60$	1.5	9	2035	8	3	60	1200
$F_o15n9s50$	1.5	9	2035	8	3	50	1200

$F_o15n11s60$	1.5	11	2035	8	3	60	1200
$F_o15n11s50$	1.5	11	2035	8	3	50	1200

---

414 **6 Conclusions**

415 Within this paper the behaviour of GCs for monopile OWTs has been investigated by means of detailed  
416 numerical computations aiming to enhance the existing knowledge by presenting robust FE models. The  
417 three-dimensional numerical models were solved employing a quasi-static approach with an explicit  
418 analysis, which was effectively used reducing the computational cost and accomplishing results of high  
419 accuracy. The HSG was modelled with the CDP model and the non-linear aspects of the material were  
420 included in the numerical models. The key parameters of the material behaviour were discussed in detail  
421 and attention was drawn to the dilation angle value, which in lack of supporting material data, is  
422 recommended to be calibrated using a sensitivity analysis.

423 The FE models were able to detect cracking of the grout core accurately along with the corresponding  
424 failure modes. Primary diagonal cracking between neighbouring shear keys was the failure mode of FE  
425 models with  $h/s = 0.05, 0.06$  whereas cylindrical failure surfaces were monitored for GCs with increased  
426 spacing between shear keys. A high level of refinement of the meshed parts is highly recommended for  
427 the shear key region in order to capture the cracks occurring in the grouted region. The notion that the  
428 grout can accommodate the load-transfer process on the connection even after cracking was also  
429 confirmed, however cracking initiation occurred at low load levels when the connection is subjected to  
430 bending.

431 The complex interacting interfaces were modelled with a Coulomb-friction model. A friction coefficient  
432  $\mu = 0.4$  yielded excellent agreement against the selected experimental data set. All models developed  
433 interface gaps at opposing sides of the connection and the results were in excellent agreement with an

434 analytical model [5]. Reflecting on the above, it can be concluded that the steel-grout bond and the  
435 damage on the grout can be modelled combining the CDP model, a Coulomb-friction model and a fine  
436 mesh in the areas of interest.

437 From the parametric study, the plain GCs which were mainly used for comparison purposes exhibited  
438 the anticipated lower performance and a pronounced de-bonding of the interface. On the other hand, the  
439 mechanical interlock provided by the shear keys led to a superior behaviour of the GCs with shear keys.  
440 The use of shear keys imposed a smoother stress distribution at the connection ends, while cracking  
441 occurred in the shear key region due to the fact that the majority of the load was transferred from the  
442 middle part of the connection. Interface opening was also found to be limited when increasing the  
443 number of shear keys.

444 Contrary to the impact of increasing shear key numbers, the higher shear key ratio did not yield superior  
445 performance owing to the change of grout failure. The relative displacement between the pile and sleeve  
446 calculated in all the numerical models revealed that shear key regions on the steel tubes occupied by  
447 shear keys is of equal importance to the height and spacing. Finally, based on the presented FE models,  
448 overlap lengths equal to unity should be avoided as the stress intensities are increased. It was also shown  
449 that the grouted length of the connection is of significant importance to the performance of the  
450 connection.

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459

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