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FATIGUE ASSESSMENT OF FULL-SCALE RETROFITTED ORTHOTROPIC BRIDGE DECKS

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Abstract: Full-scale fatigue tests were performed on two retrofitted orthotropic bridge decks (OBD).

The retrofitting systems consist of adding a second steel plate on the top of the existing deck. The aim is to reduce the stresses at the fatigue sensitive details and therefore extend the fatigue life of the OBD by stiffening the existing deck plate. Two retrofitting systems have been studied. The bonded system consists in bonding a second steel plate to the existing deck by vacuum infusing a thin adhesive layer (2 mm) between the two steel plates. The sandwich system consists in bonding the second steel plate through a thick polyurethane core (15 mm). The aim of the study was to assess the fatigue performance of both retrofitting. No fatigue damage was detected in the retrofitting layers during fatigue tests after 3 million cycles of wheel load. The stresses close to the deck-plate-to-stiffener welds reduce by at least 55% when using the bonded steel plates system and 45% when using the sandwich steel plates system. Both systems proved to have sufficient fatigue life to withstand traffic wheel loads running on orthotropic bridge decks and help extending the fatigue life of the existing OBD.

Key Words: *Orthotropic steel bridge decks, Reinforcement, Fatigue, Structural bonding, Sandwich structures*

INTRODUCTION

Orthotropic steel bridge decks (OBDs) are largely used in most of the major long span bridges in the world due to their low dead-weight at an attractive cost. However, in the past decades, severe fatigue cracks have been reported at several welded joints in OBDs (Jong 2004, Fisher 2016). One of the most threatening is the one that grows through the deck plate at the longitudinal welds between the deck plate and trapezoidal stiffener (deck-plate-to-stiffener weld) (Ya et al 2011). These welds have very high stress concentrations particularly at the intersection with the crossbeams (Kolstein, Wardenier, and Weijde

34 1998). The major reason for these fatigue cracks is the low stiffness of the deck plate, which is insufficient to deal with the
35 heavy traffic wheel loads (Miki 2006). Moreover, the increase of heavy traffic in the past decades makes these fatigue
36 phenomena an even greater concern.

37 Several research projects studied different renovation systems to strengthen existing OBDs. The common idea is to
38 substitute the existing asphalt wearing course by a stiffer overlay. Research has been done on replacing the wearing course by a
39 reinforced concrete overlay (Walter 2005, Jong 2006, Zhang et al 2016). Field measurements performed during renovations of
40 several orthotropic bridges in the Netherlands, where the common 50 mm thick asphalt surface was replaced by 50 mm thick
41 reinforced concrete overlay, showed a stress reduction close to the welds of 80% after the reinforcement when compared with
42 no surfacing (Kolstein and Sliedrecht 2008). Alternative wearing courses solutions to the classic asphalt layer have been also
43 proposed by Medani (2006). Other retrofitting techniques focus on delaying the crack propagation by acting directly at the
44 welded areas, such as Impact Crack-closure Retrofit (ICT), by inducing compressive residual stresses introduced by plastic
45 deformation through high-speed impact (Yamada et al 2015, Zhiyuan et al 2016).

46 However, most of the mentioned alternatives for replacing the existing wearing course are often too heavy for application
47 on existing movable bridges with orthotropic decks. For these structures, the weight limits are very strict and light-weight
48 retrofitting overlays are the only possible solution. Previous studies have suggested light weight retrofitting systems which
49 consist of adding a second steel plate to the existing deck. The second (new) steel plate is bonded to the existing deck either by
50 vacuum infusing a 2 mm thick adhesive layer between the two steel plates (Labordus 2006) – referred in this paper as bonded
51 steel plate system, or by a 15 to 30 mm thick polyurethane core – referred in this paper as sandwich steel plate system (Vincent
52 and Ferro 2004). Both retrofitting solutions are regarded as lightweight - between 50 and 80 kg/m². Previous research

53 performed on beams representing the mentioned retrofitting overlay systems show stress reduction factors between 60% and
54 75% at the existing deck plate after retrofitting (Teixeira de Freitas et al., 2010, 2011, 2013a). Structural monitoring of a pilot
55 application of the bonded system with 6 mm thick second steel plate retrofitting a 12 mm thick existing deck plate of the
56 Scharsterrijn movable bridge in the Netherlands showed 55% stress reduction at the deck-plate-to-stiffener weld after
57 renovation (Teixeira de Freitas et al 2012a). Further application of the same system in Hartelkanaal 12 mm thick existing deck
58 plate with 10 mm thick second steel plate promised an additional fatigue service life to the existing deck of 40 years (Voermans
59 et al 2014). A trial full-scale application of the sandwich steel plate system also showed significant reduction in the deck
60 deflection and the additional advantages in terms of thermal insulation and decreased noise emission (Feldmann et. al 2007).
61 Full scale static tests of the steel-polyurethane OBD showed stress reduction from 40% to 80% close to the fatigue-sensitive
62 details and a good performance under compressive longitudinal stresses (Teixeira de Freitas et al 2013b, Shan C. and Yi Y.
63 2016, 2017, Shan C 2017). Nevertheless, the retrofitting systems also have to be evaluated in terms of fatigue performance.
64 When applying a reinforcement system to a fatigue cracked OBD, it is important to guarantee that the reinforcement system
65 will not raise new fatigue problems to the structure.

66 In this paper, full scale fatigue test are performed on OBD reinforced with bonded steel plate system and sandwich plate
67 system in order to evaluate and compare their fatigue performance under full-scale. The aim is to determine the fatigue life of
68 both retrofitting systems when subject to realistic wheel loads.

69 **BRIDGE DECK SPECIMENS**

70 Two orthotropic deck-panels were manufactured with the same geometry: one was retrofitted using the bonded system
71 and another using the sandwich system.

72 Figure 1 shows a drawing of the deck specimens. The specimens were 5000 mm long and 2000 mm wide. The deck plate
73 was 12 mm thick and it was reinforced by three longitudinal trapezoidal stiffeners (Krupp profile FHK 2/325/6: height of 325
74 mm, a base distance between the outer side of the trough legs of 300 mm, bottom width of 105 mm and a plate thickness of 6
75 mm) and two transverse crossbeams 3000 mm apart. In a real bridge, the traffic runs in the longitudinal direction on top of the
76 deck plate. The deck is made of steel grade S355 ($f_y = 355$ MPa, $f_u = 510$ MPa, $E = 210$ GPa, $\nu = 0.3$) (EN 1993-1-1 2006).

77 Figure 2 shows the nominal thicknesses of the bonded and sandwich retrofitting systems. In the bonded system, a 6 mm
78 thick second steel plate was bonded to the existing 12 mm thick deck plate with a 2 mm thick adhesive. In the sandwich system,
79 a 5 mm thick second steel plate was bonded to the 12 mm thick existing deck plate with a 15 mm thick polyurethane core.

80 The second steel plate is made of steel grade S355. The adhesive is an epoxy paste resin - Epikote resin EPR 04908 with
81 hardener Epikure curing agent EPH 04908 (properties at room temperature: $E_t = 2929$ MPa; $\sigma_{\max} = 69$ MPa; $\nu = 0.4$ (Teixeira
82 de Freitas, Kolstein and Bijlaard 2010)). The core material is a polyurethane with density 1150 kg/m^3 (properties at room
83 temperature: $E_t = 721$ MPa; $\sigma_{\max} = 25$ MPa; $\nu = 0.36$ (Teixeira de Freitas, Kolstein and Bijlaard 2011)).

84 The manufacturing procedure of the bonded system consisted on the following chronological steps: (1) grit blast and clean
85 the steel surfaces (Sa 2 1/2 - ISO 8501 (2007)); (2) apply a primer on the steel surfaces to be bonded; (3) glue spacers with
86 thickness of 2 mm; (4) place the new steel plate on the top of the existing deck; (5) prepare the cavity between the steel plates
87 for infusion; (6) vacuum infuse the adhesive and (7) cure during 16 hours between 40°C and 50°C .

88 The manufacturing procedure of the sandwich system consisted on the following chronological steps: (1) grit blast and
89 clean the steel surfaces (Sa 2 1/2 - ISO 8501 (2007)); (2) weld steel bars with the core thickness on the perimeter of the existing
90 deck plate; (3) glue PU spacers with the core thickness; (4) place the new steel plate on the top of the perimeter bars and weld

91 through the perimeter; (5) inject the PU into the cavity between the steel plates and (6) cure at room temperature during 48 h.

92 **Instrumentation**

93 Strain gauges were glued to the bottom side of the deck, at three cross-sections: two crossbeams (A and B) and midspan
94 between both crossbeams - see Figure 1. The location of the strain gauges is shown in Figure 3.

95 The position of the strain gauges was the same for both deck specimens. In order to apply the strain gauges inside the
96 troughs, parts of the troughs were cut out and re-welded again.

97 The strain gauges were positioned to measure transverse strains, except the ones at the bottom of the stiffeners at the
98 midspan, which measure longitudinal strains. The exact location of the strain gauges close to the deck-plate-to-stiffener welds
99 is shown in Figure 4.

100 **EXPERIMENTAL PROCEDURE**

101 The deck specimens were loaded with wheel prints type C at the crossbeam cross-sections and at midspan between
102 crossbeams, in accordance with the fatigue load models of EN 1991-2 (2003). Wheel print type C is a single-tyre 320 mm long
103 and 270 mm wide, usually called super-single.

104 In total seven fatigue test were performed on each deck specimen. At the crossbeams, six fatigue tests were performed in
105 total, one on each trough-to-crossbeam joint (2 crossbeams x 3 troughs). Wheel type C was aligned with the crossbeam's web
106 and with each trough. Figure 5 shows one example of a fatigue test performed at the crossbeam A (trough 2-to-crossbeam A
107 joint). At midspan between crossbeams, one fatigue test was performed with wheel type C aligned with the middle trough. The
108 seven fatigue tests on each deck specimen were performed one by one. The same load cases were used for both deck
109 specimens. Prior to the fatigue tests, static tests were performed with the same wheel print and location as the fatigue tests.

110 The bottom flanges of the two crossbeams were clamped to the ground. Figure 6 shows a photo of the test set up. The
111 load was applied on the deck through a 30 mm thick steel plate and 30 mm thick rubber plate, with the rectangular area of the
112 wheel print type C.

113 The fatigue tests were carried out under load control with a constant applied load ratio $R = 0.1$ ($R = P_{\min} / P_{\max}$). The wave
114 form was sinusoidal. The bonded steel plates specimen was loaded at a frequency of 7 and 5 Hz at the crossbeam
115 cross-sections and at midspan between crossbeams, respectively. The sandwich steel plates specimen was loaded at a
116 frequency of 2 Hz at both locations. Previous research on sandwich beam specimens has shown that for higher frequencies
117 than 2 Hz, the temperature of the PU core increases with fatigue cycling (Teixeira de Freitas, Kolstein and Bijlaard 2013a). In
118 order to avoid this undesirable thermal effect, the frequency was kept to 2 Hz on the sandwich specimens.

119 The tests were performed at three load levels. The maximum loads P_{\max} were 160 kN, 110 kN and 90 kN ($\Delta P = 144$ kN,
120 99 kN and 81 kN, respectively). At the crossbeams, two tests were performed at each load level. At midspan between
121 crossbeams, the bonded steel plates specimen was tested at $P_{\max} = 160$ kN and the sandwich steel plates at $P_{\max} = 110$ kN.
122 Chronologically, the bonded steel plates specimen was tested first, and when tested at midspan between crossbeams, a fatigue
123 crack appeared in an early stage at the weld of the trough made for the instrumentation holes. The test had to be stopped, the
124 trough was cut out and replaced by larger piece in order to reduce the stresses at the welds. After re-welding the new
125 trough-piece, the test was restarted. In order to avoid this problem on the sandwich steel plates specimen, the maximum load
126 level at midspan between crossbeams was decreased to 110 kN. No fatigue crack was detected at the trough of this specimen.

127 The load levels used in the fatigue tests are higher than the ones recommended at the fatigue load model 2 of EN1991-2
128 (2003). In the fatigue model, the maximum load of wheel type C is 60 kN. The loads used in the fatigue tests are from 1.5 to

129 2.67 times higher than the ones described by the fatigue model.

130 Ultrasonic Non-Destructive-Testing A-scan was performed at the loaded areas of the bonded steel plates reinforced
131 specimen. The aim was to detected delamination areas of the adhesive layer. The scanning was performed before and after the
132 fatigue tests, and every million cycles.

133 **FINITE ELEMENT ANALYSIS**

134 A Finite Element Analysis was performed to determine the stress fields in the OBD during testing. The geometry, wheel
135 loads and boundary conditions of the model were simulating the full-scale test. The commercial FEA program ABAQUS was
136 used to run the simulations.

137 The geometry of the model follows the nominal dimension of the several parts of the OBD. The welds were modeled
138 only as the geometrical connection between the deck plate, stiffeners and cross beams. The linear elastic mechanical properties
139 of the materials mentioned previously were used. Figure 7 shows a 3D view of the model. Only half of the deck plate has
140 been model and a symmetry boundary condition was applied at midspan. More details on the numerical work, including mesh
141 details, element type, number of nodes and mesh convergency study can be found in Teixeira de Freitas (2012b, 2013b).

142 **RESULTS**

143 **Static tests and numerical validation**

144 In order to validate the numerical results, the strains measured by the strain gauges during the static tests are compared
145 with the strains from the FEM. Figure 8 shows the transverse strain values measured during a static test with 100 kN wheel
146 type C positioned at the cross beam location, both for the bonded system and the sandwich system. The transverse strains on
147 the bottom side of the deck plate measured by the strain gauges at the crossbeam and at 75 mm from the crossbeam are

148 compared with the numerical strains from the FEM at the same locations. Figure 9 shows the results of a static test of 100 kN
149 wheel type C positioned midspan between crossbeams. Also here, the strain measured by the strain gauges are compared with
150 the numerical strains.

151 For both reinforcement systems, the strains measured close to the deck-plate-to-stiffener welds are higher at the cross
152 beam location than at midspan. This stress concentration is caused by the stiffness singularity introduced by the cross beam
153 web. Also important to notice is that, at the cross beam location, the non-loaded troughs experience an insignificant strain when
154 the adjacent trough is loaded. This shows that the fatigue test results at the crossbeam location at the three different troughs can
155 be treated as separate tests.

156 For both reinforcement systems and load locations, the numerical results correspond well with the experimental results.

157 **Fatigue tests**

158 Figure 10 shows a selection of strain ranges versus the number of cycles, measured on the bonded steel plates reinforced
159 deck. Figure 10a shows the results of eight strain gauges close to a deck-plate-to-stiffener weld in one of the crossbeam fatigue
160 tests. The maximum load in this example was 160 kN. Strain gauge SG05, SG06 and SG 07 are close to the weld root, SG01
161 and SG03 are close to the weld toe and SG09, SG10 and SG11 are between stiffeners webs. From all gauges at this location,
162 the one with the biggest change during testing is strain gauge SG06. This strain gauge is aligned with the crossbeam web, close
163 to the weld root. The range started to decrease in an early stage of the fatigue tests. As a response to that decrease, strain range
164 SG01 and SG03 started to increase. This is a consequence of the stress redistribution due to the local stiffness loss close to
165 strain gauge SG06. The fatigue tests at lower load levels showed a similar strain pattern but at lower magnitudes. Figure 10b
166 shows the strain ranges measured during the fatigue test at midspan on the deck reinforced with the bonded system. Strain

167 gauge SG8 and SG9 are close to the deck-plate-to-stiffener weld root, SG3 and SG4 are close to the weld toe, and SG12 and
168 SG13 are between stiffeners' webs. The maximum load level was $P_{max} = 160$ kN. There were no significant changes during
169 testing in any of the strain gauges results. The main difference between the tests at the crossbeam (Figure 10a) and at midspan
170 (Figure10b) is the strain close to the welds, which are significantly lower at midspan than at crossbeam location (at midspan
171 SG9 is less than 100μ , while at the crossbeam SG06 is approximately 500μ). The crossbeam's web is a point of very high
172 stiffness which leads to high stress concentration. The strain between the stiffener webs is higher at midspan between
173 crossbeams than at the crossbeam (SG13 is approximately 900μ at midspan, while SG10 is 600μ at the crossbeam).

174 The ultrasonic NDT performed before and after the fatigue tests didn't detect any change in the integrity of the adhesive
175 layer in none of the locations. This means that there was no delamination in the adhesive layer caused by the fatigue loading.

176 Figure 11 shows the strain ranges measured on the sandwich steel plates reinforced deck for the same selection of strain
177 gauges. In general, the strain ranges measured at the sandwich steel plates deck specimen have a very similar pattern to the
178 corresponding ones measured at the bonded steel plate deck specimen. In Figure 11a, the biggest change in the strain range
179 occurs at the same deck location, strain gauge SG06. The range also started to decrease in an early stage of the fatigue tests. The
180 major difference between the bonded and sandwich reinforced decks is the strain magnitude of SG06 at the maximum load
181 level ($P_{max}=160$ kN). In the bonded reinforced deck the initial strain range is approximately 550μ while in the sandwich
182 reinforced deck the initial strain range is approximately 725μ . Figure 11b shows the strain ranges measured during the fatigue
183 test at midspan on the deck reinforced with the sandwich system. The maximum load was $P_{max}=110$ kN. There were no
184 significant changes during testing in any of the strains measured. As for the bonded reinforced deck, the strains measured close
185 to the welds are considerably higher at the crossbeam location than at midspan for the same load.

186 The ultrasonic A-scan was not performed on the sandwich steel plates deck specimen, since after a trial test on a reference
187 sandwich panel, it was concluded that the damping of the sound wave when crossing the interface between the steel plate and
188 the PU core material was as high as when crossing an interface between steel plate and air. Therefore, no distinction could be
189 made between good and bad adhesion quality at the interface between the steel and the core.

190 As no major failure was detected in any of the fatigue tests, all tests were stopped after approximately 3 million cycles.

191 During and after the fatigue tests, both bridge deck specimens were visually inspected for fatigue cracks at the welds. At
192 the crossbeam location, several fatigue cracks were observed at the deck-plate-to-stiffener welds. Figure 12 shows pictures of
193 the fatigue cracks close to those welds on both reinforced deck-panels. These pictures were taken after cutting a part of the deck
194 specimens at the crossbeam. At midspan no cracks were observed.

195 The strain gauges showing major changes are always close to the deck-plate-to-stiffener welds, exactly where fatigue
196 cracks were observed. No delamination was detected in the adhesive layer by the ultrasonic NDT. Therefore, it can be
197 concluded that the decrease of strain range close to the welds is caused by fatigue crack initiation at the deck-plate-to-stiffener
198 weld, and not by fatigue damage of the reinforcements. Although at the sandwich steel plates deck specimen, there was no
199 NDT inspection to the interface between the core and the steel plate, the fact that the corresponding strain range pattern is very
200 similar to the bonded steel plates deck panel and that fatigue cracks were also observed at the deck-plate-to-stiffener welds, it
201 can be concluded that: there was no fatigue damage on the sandwich steel plates reinforcement.

202 It is important to remember that the main objective of the full-scale fatigue tests was to evaluate the fatigue behavior of the
203 retrofitting systems, rather than the fatigue life of the welds on an OBD. As no delamination was detected in the adhesive layer
204 in none of the fatigue tests, it can be considered that the seven fatigue tests on the bonded steel plates retrofitted deck were run

205 out test (no fatigue failure). The same can be said for the sandwich steel plate retrofitted deck as there was no indication of
206 delamination in the sandwich overlay in none of the seven fatigue tests.

207 Table 1 summarizes the fatigue results of the deck-plate-to-trough weld on both retrofitted deck specimens. The fatigue
208 life n_f is based on the strain ranges measured by the gauges close to the deck-plate-to-stiffener weld roots, aligned with the
209 crossbeam or with midspan between crossbeams (SG06 at the crossbeam and SG09 at midspan). The results are presented for
210 two different failure criteria: 10% and 25% strain fall. These failure criteria were used by (Kolstein 2007) to define the fatigue
211 design classification of this type of fatigue crack (cracks at deck-plate-to-stiffener weld that grow through the deck-plate
212 thickness). The main difference is that (Kolstein 2007) used strain gauges on the top side of the deck plate and, in this study,
213 strain gauges on the bottom side of the deck-plate were used. At each fatigue test, two deck-plate-to-stiffener welds were tested
214 simultaneously and, therefore, the fatigue results are presented for both welds (one at each side of the stiffener). When no
215 changes were observed in the measured strain ranges during fatigue tests and no cracks were detected, the tests were
216 considered run-out tests.

217 **DISCUSSION**

218 In this section, the results of the full-scale fatigue tests are discussed mainly to evaluate the fatigue life of the retrofitting
219 systems. The results are compared with SN curves proposed in previous research. In order to make this comparison, it is
220 important to know the stress distribution in the retrofitting layers during the full-scale testing. This distribution was determined
221 by Finite Element Analysis (FEA) of the full-scale tests already presented.

222 Since several fatigue cracks were found during fatigue testing, a brief analysis of the fatigue life of the welded joints of the
223 retrofitted deck is presented in the end of this section.

224 **Stress distribution in the retrofitting systems**

225 Previous research in beam specimens showed that the main fatigue failure mode of the bonded steel plates system when
226 subject to four-point bending tests (bending and shear location) is adhesive shear failure (Teixeira de Freitas, Kolstein and
227 Bijlaard 2013a). For the same tests performed in the sandwich steel plates system, the main fatigue failure observed was
228 delamination between the steel face and the core. Therefore, the fatigue behavior of the bonded and sandwich retrofitting
229 systems depends on the shear stress in the adhesive layer and at the interface between the steel face and the core, respectively.

230 The shear stress distribution in the reinforcement systems during the full scale test was determined by the FEA. Figure 13
231 shows one example of the shear stress distribution τ_{xy} along the width of the deck in the adhesive layer (Figure 13a) and in the
232 interface between the steel plate and the core (Figure 13b). τ_{zy} was neglected since the values were significantly lower than τ_{xy} .
233 Two load cases are presented: wheel type C at midspan between crossbeams and wheel type C at the crossbeam cross section.
234 On both cases the wheel is aligned with the middle trough.

235 The maximum shear stress in the adhesive layer of the bonded steel plates system occurs at $x=900$ mm and $x=1100$ mm,
236 in between the stiffeners webs (Figure 13a). For 100 kN wheel load, the maximum shear stress is approximately 8 MPa at the
237 crossbeam cross section and 7 MPa at midspan between crossbeams. Figure 13b shows the results of the shear stress at the
238 steel-plate-core interface of the sandwich steel plates system. The maximum values between the two interfaces with the steel
239 plates were taken. The maximum shear stress also occurs at $x=900$ mm and $x=1100$ mm. For 100 kN wheel load, the
240 maximum shear stress is approximately 2.3 MPa at the crossbeam cross section and 2.1 MPa at midspan between crossbeams.

241 **Fatigue behavior of the retrofitting systems**

242 The end of the fatigue life of the bonded steel plates system should be taken at the moment when delamination occurs in

243 the adhesive layer. As no delamination was detected after any of the full-scale fatigue tests, it is concluded that no fatigue
244 damage occurred on the bonded steel plates system during full-scale fatigue testing. The same can be considered for the
245 sandwich steel plates system as no delamination was detected during any of the full-scale fatigue tests.

246 Figure 14 shows the stress-cycle SN diagrams for both reinforcements. In Figure 14a, the fatigue life n_f of each fatigue test
247 performed in the bonded steel plates retrofitted deck, is plotted against the shear stress range at the adhesive layer $\Delta\tau_{ad}$. The
248 shear stress range is the maximum shear stress, presented in Figure 13a, multiplied by the amplitude load in the correspondent
249 fatigue test ($\Delta P = 0.9 \cdot P_{max}$). Figure 14b plots the fatigue life n_f of each fatigue test, performed in the sandwich steel plates
250 retrofitted deck, against the shear stress range at the interface between core and steel plate $\Delta\tau_c$. The shear stress range is the
251 maximum shear stress, presented in Figure 13b, multiplied by the amplitude load in the correspondent fatigue test. The results
252 from the full-scale tests are plotted together with fatigue results obtained in previous research from four-point bending fatigue
253 tests on both retrofitting systems (Teixeira de Freitas, Kolstein and Bijlaard 2013a).

254 For the bonded steel plates system, the shear stress at the full-scale tests is close to the fatigue threshold of the adhesive
255 layer proposed in Teixeira de Freitas (2013a), approximately 8 MPa. For the sandwich steel plates system, the shear stress at
256 the interface on the full-scale tests is lower than the fatigue threshold proposed in Teixeira de Freitas (2013a), approximately 4
257 MPa. Therefore, the results from the full-scale fatigue tests are in agreement with the SN curves of both reinforcement systems
258 proposed by Teixeira de Freitas (2013a).

259 **Fatigue life of the welded joints of the retrofitted deck**

260 Although it was not the main objective of the full-scale test, a brief analysis of the fatigue life of the welded joints of the
261 retrofitted bridge decks is presented in this section since several fatigue cracks were found during fatigue testing.

262 As shown in Figure 12, the crack found at the crossbeam location is the well-known fatigue crack at the
263 deck-plate-to-stiffener weld. The crack starts at the root of the weld between the longitudinal stiffener and the deck plate, at the
264 point where it intersects with crossbeam web. The crack grows through the thickness of the deck plate, from the bottom to the
265 top side of the plate (Kolstein 2007).

266 Figure 15 shows the SN results of the welds at the crossbeam location, based on both criteria described in Table 2. The
267 stress range $\Delta\sigma$ is determined at the point where the crack initiates, which means at the deck-plate-to-stiffener weld root on the
268 bottom side of the deck plate. For this typical detail, as the stress gradient close to the weld is very high, the stress is determined
269 based on the geometrical stress range - hot spot method. The hot spot method is recommended by Hobbacher (2009), for
270 fatigue assessment of general welded joints and, by Kolstein (2007) for this specific fatigue crack of orthotropic steel bridge
271 decks. The method consists in extrapolating the structural stress from two measuring points to where the crack initiates, called
272 hot spot point. The two measuring points are $0.4t$ and t from the hot spot point, t being the deck plate thickness (12 mm). The
273 stress at the measuring points were taken from the FEA of the full-scale tests. The fatigue life of the welds is compared with the
274 fatigue strength SN curve defined at EN1993-1-9 (2005). The detail category 125 is the one recommended by Kolstein (2007)
275 for these types of fatigue cracks.

276 The fatigue results of the welds in the bonded and sandwich retrofitted decks follow the same tendency when considering
277 the same criteria for the fatigue life. The fatigue life of the welds in the sandwich retrofitted deck is slightly longer than in the
278 bonded retrofitted deck. The slope of the fatigue results is closer to the fatigue strength of the 125 detail category when the 25%
279 strain fall failure criterion is used. However, the results are worse than expected, since the detail category 125 should give
280 conservative fatigue strength of the fatigue life of these welds. This is related with the fact that, in this study the failure criteria

281 are based on strain falls measured at the bottom side of the deck plate, very close to the weld root. Kolstein (2007) based his
282 recommendation on strain falls measured at the top side of the deck plate, and therefore farther away from the weld root. As
283 this type of crack initiates at the weld root, the strain measured in this study are much more sensitive to the crack initiating at the
284 weld root than the ones used by Kolstein (2007). Therefore the strain fall occurs earlier in the strain gauges used in this study (at
285 the bottom side of the deck plate) than in the ones used by Kolstein (2007) (at the top side of the deck plate).

286 From extrapolation of these SN curves, one can predict the fatigue life of the welds at any stress range present at the weld
287 root, in a bonded or sandwich steel plates retrofitted deck. This means that, if one can determine the stress reduction at the weld
288 root after the retrofitting, one can predict the improvement in the fatigue life of the weld just by using the stress reduction
289 factors on these SN curves.

290 Figure 16 shows the SN results of the welds at midspan between crossbeams. As no fatigue cracks were found, all results
291 are run-out tests. The stress range was determined by the nominal stresses at the deck plate at the stiffener web location. The
292 results are compared with the details category for the fatigue strength of these welds recommended by Kolstein (2007) (125
293 detail category) and by at EN1993-1-9 (2005) (71 detail category). The results are below the constant amplitude fatigue limit
294 ($\Delta\sigma$ at 5 million cycles) of the 125 detail category, which explains the absence of fatigue cracks at this location.

295 **INFLUENCE OF THE RETROFITTING THICKNESS – PARAMETRIC STUDY**

296 Based on the previous analysis, one can conclude that the fatigue assessment of a retrofitting system for OBD can be
297 performed by:

- 298 (1) Determine the Stress Reduction Factor (SRF) at the critical welded joints of the bridge deck;
- 299 (2) Determine the fatigue threshold of the retrofitting system.

300 The first one will allow to predict the increase of the fatigue life span of the OBD, and the second one will guarantee that the
301 retrofitting system has a longer fatigue life than the bridge deck. In this section this analysis will be performed to different
302 thickness of the bonded and sandwich steel plates retrofitting system from the ones tested in the full-scale bridge decks.
303 Therefore the influence of the thickness on the efficiency of the retrofitting system will be analyzed. The FEA mentioned
304 earlier in this paper will be used to simulated the different retrofitting scenarios. The load condition simulated was 100kN
305 wheel type C aligned with the middle trough and positioned either at the crossbeam (see Figure 5) or at midspan between
306 crossbeams.

307 Table 2 shows the different retrofitting system simulated. For the bonded steel plates system, the adhesive thickness was kept 2
308 mm, since this is the nominal value to be applied in actual retrofitting, and the thickness of the second steel plate was varied
309 between 6 mm and 12 mm. For the sandwich steel plates systems, the thickness of the core was varied between 15 mm and 30
310 mm, and the thickness of the second steel plate was varied between 5 mm and 8 mm. The retrofitting weight is also shown as a
311 comparative parameter between the different solutions. For all cases studied, the thickness of the existing deck plate is 12 mm.

312 **Stress reduction factor**

313 The SRF was determined using equation (1). The SRF was determined at four deck locations -- see Figure 17. The values were
314 determined at the crossbeam location and at midspan between crossbeams.

$$315 \quad SRF = 1 - \frac{\sigma_{retrofitted\ deck}}{\sigma_{un-retrofitted\ deck}} \quad (1)$$

316 Figure 18 and 19 show the SRF as a function of the weight for the bonded and sandwich steel plates system, respectively. SRFs
317 higher than 100% occur when the stress value changes the signal from negative to positive, or the other way around. The SRF
318 of details I, II and III gives an indication of the retrofitting performance at the transverse stresses due to local bending of the

319 deck plate.

320 For the bonded steel plates system B.12.2.6 tested in the full-scale bridge decks, the transverse stresses close to the
321 deck-plate-to-stiffener welds (group II and III) reduce approximately 55% to 60% at the cross beam location after the
322 retrofitting, and about 70% to 90% at the midspan between crossbeams. At the same locations, the sandwich steel plates
323 system S.12.15.5, the stresses reduce approximately 45% at the crossbeam location and 50% to 60% at the midspan between
324 crossbeams. The least affected stresses are the longitudinal stresses at the bottom of the stiffeners (Group IV). The stress are
325 reduced by 20% in the B.12.2.6 solutions and by 30% in the S.12.15.5 solutions.

326 Concerning details I, II and III both at the crossbeam and at midspan between the crossbeam, the results show that: increasing
327 the thickness of the second steel plate of the bonded steel plates reinforcement by 2 mm adds on average 6% to the SRFs; each
328 increase of 5 mm of core thickness of the sandwich steel plates adds on average 3% to the SRFs. Increasing the thickness of the
329 second steel plate of the sandwich steel plates from 5 mm to 8 mm adds 7% to the SRFs.

330 Comparing the two retrofitting systems with the same weight, for details I, II and III at the crossbeam location, the SRFs are
331 higher when using the bonded steel plates than when using the sandwich steel plates. The sandwich steel plates system can
332 achieve SRF similar to the ones of the bonded steel plates system but needs double the weight.

333 At midspan between crossbeams, the SRF of details II and III increases significantly when compared to the ones at the
334 crossbeam location, especially that of detail III of the bonded steel plates.

335 Also at midspan between crossbeams when comparing two systems with the same weight, details II and III have higher SRF in
336 the bonded steel plates system than in the sandwich steel plates system. The SRF of detail I are similar in the sandwich steel
337 plates reinforcement and in the bonded steel plates systems.

338 For detail IV, the sandwich steel plates system performs better than the bonded steel plates system. This detail gives an
339 indication of the global effect of the reinforcement (longitudinal stress due to global bending of the OBD).

340 Figure 19 (b) shows that the SRF of the solution S.12.15.8 (80 kg/m²) at midspan between crossbeams is slightly out of the
341 tendency. The overall tendency of the graph gives an idea of the effect of the core thickness, while the S.12.15.8 gives an
342 indication of the effect of the second steel plate thickness. The results indicate that this effect is positive for detail III and
343 negative for details I and IV.

344 Overall, the bonded steel plates system has a good performance in reinforcing the structure locally, as for example close to the
345 deck-plate-to-stiffener welds. The sandwich steel plates system is more a global reinforcement. It affects not only the local
346 stresses, but also the global stresses. The sandwich steel plates systems improves its performance, when the existing steel deck
347 becomes flexible (less stiff), and the bending of the deck becomes larger. This is the case in detail I and detail IV at midspan
348 between crossbeams.

349 **Fatigue life of the retrofitting system**

350 Besides extending the fatigue life of the welds, the reinforcements should not give rise to new fatigue problems. Therefore it is
351 important to evaluate their fatigue life.

352 The full-scale fatigue tests showed no fatigue damage in the retrofitting systems. This result could have been predicted based
353 on the SN diagrams of each retrofitting system obtained from the fatigue tests on retrofitted beams – see Figure 14. The
354 maximum shear stresses values in the adhesive layer and in the core during the fatigue full scale tests were in the vicinity or
355 below the fatigue thresholds of those SN diagrams.

356 Table 3 shows the maximum shear stress τ_{xy} at the adhesive layer and at the steel-core interface obtained from the FEA of the

357 bonded and sandwich steel plates system, respectively. The values correspond with a 100 kN wheel load type C aligned with
358 the stiffener either at the crossbeam location or at midspan between crossbeams.

359 The retrofitting solutions tested on the full-scale fatigue tests, B.12.2.6 and S.12.15.5 for the bonded and sandwich respectively,
360 have the highest values of shear stress. Therefore, if no fatigue damage was observed during the fatigue tests performed on
361 those retrofitting solutions, no fatigue damage is expected to occur in all the other reinforcements with lower shear stresses.

362 CONCLUSIONS

363 The fatigue life of full-scale retrofitted orthotropic bridge deck specimens was investigated. Two retrofitting solutions
364 were compared which consisted in bonding a second steel plate to the existing deck either using a thin 2 mm thick adhesive
365 layer (bonded system) or a thick 15 mm thick polyurethane core (sandwich system). The retrofitted deck specimens were
366 cyclically loaded by single tire wheel prints at the crossbeam cross section and at midspan between crossbeam. From the
367 analysis of the full-scale tests, the following conclusions can be drawn:

- 368 • The fatigue threshold of the retrofitting systems determined on beam tests under bending is valid for the fatigue
369 life of the retrofitting system on full-scale OBD under wheel loads.
- 370 • Under maximum wheel loads between 160 kN and 90 kN, the stresses at the retrofitting systems are lower than
371 their fatigue threshold. The retrofitting solutions proved to have sufficient fatigue life to withstand traffic wheel
372 loads running on orthotropic bridge decks, without fatigue damage.
- 373 • The fatigue evaluation of a retrofitting system for OBD can be performed by determining the Stress Reduction
374 Factor (SRF) at the critical welded joints of the bridge deck and the fatigue threshold of the retrofitting system.
- 375 • Using the bonded system, the transverse stresses close to the deck-plate-to-stiffener weld reduce by at least 55%

376 at the crossbeam location and 70% at midspan between crossbeams. Each 2 mm added to the thickness of the
377 second steel plate will reduce the stresses 6% further. Using the sandwich system, the same stresses are reduced
378 by at least 45% at the crossbeam location and 55% at the midspan between crossbeams. Each 5 mm added to
379 the sandwich core thickness will increase the stress 3% further.

- 380 • For similar weights, the bonded steel plates system is more efficient in reducing the local stresses close to the
381 welds while the sandwich steel plates system is more efficient in reduction the global stresses of the bridge
382 deck.

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387 **REFERENCES**

- 388 EN 1993-1-1 (2006), *Eurocode 3: Design of steel structures Part 1-1: General Rules and rules for buildings*, CEN.
- 389 EN 1991-2 (2003), *Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges*, CEN.
- 390 EN 1993-1-9 (2005), *Eurocode 3: Design of steel structures Part 1-9: Fatigue*, CEN
- 391 Feldmann M, Sedlacek G, Geßler A. (2007). “A system of steel-elastomer sandwich plates for strengthening
392 orthotropic bridge decks”, *Mechanics of Composite Materials*, 43(2): 183–190.
- 393 Fisher, J. and Barsom, J. (2016), “Evaluation of Cracking in the Rib-to-Deck Welds of the Bronx–Whitestone Bridge,
394 *Journal of Bridge Engineering*, 21(3): 04015065.
- 395 ISO 8501 (2007), *Preparation of steel substrates before application of paints and related products*, CEN.
- 396 Hobbacher, A. (2009). “The new IIW recommendations for fatigue assessment of welded joints and components - A
397 comprehensive code recently updated”, *International Journal of Fatigue*, **31** (1), 50 – 58.
- 398 Jong de, F. (2004). “Overview fatigue phenomenon in orthotropic bridge decks in the Netherlands”, In *2004*
399 *Orthotropic Bridge Conference*, Sacramento, USA
- 400 Jong de, F. (2006). *Renovation techniques for fatigue cracked orthotropic steel bridge decks*. PhD thesis, Delft
401 University of Technology, Netherlands.
- 402 Kolstein, H., Wardenier, J., and Weijde, H. (1998). “A new type of fatigue failures in steel orthotropic bridge decks”, In
403 *Fifth Pacific Structural Steel Conference*, Seoul, Korea.

404 Kolstein, H. (2007). *Fatigue classification of welded joints in orthotropic steel bridge decks*. PhD thesis, Delft
405 University of Technology, Netherlands.

406 Kolstein, M. and Sliedrecht, H. (2008). “Reduction of traffic induced stresses using high strength concrete”. In *2008*
407 *International Orthotropic Bridge Conference*, Sacramento, USA.

408 Labordus M. (2006). “Vacuum infused bonded steel reinforcing plates for bridge rehabilitation.” In: *International*
409 *bridge technology conference and trade show*, Rotterdam, The Netherlands.

410 Liu, R., Liu, Y., Ji, B. Wang, M. and Tian, Y. (2014), “Hot spot stress analysis on rib–deck welded joint in orthotropic
411 steel decks”, *Journal of Constructional Steel Research*, 97, 1-9.

412 Medani, T. (2006). *Design principles of surfacings on orthotropic steel bridge decks*. PhD thesis, Delft University of
413 Technology, Netherlands.

414 Miki, C. (2006). “Fatigue damage in orthotropic steel bridge decks and retrofit work”, *Steel structures* **6**, 255–267.

415 Shan C. and Yi Y. (2016). “Stress concentration analysis of an orthotropic sandwich bridge deck under wheel loading”,
416 *Journal of Constructional Steel Research*, 122: 488–494

417 Shan C. and Yi Y. (2017). “An experimental and numerical study on the behavior of a continuous orthotropic bridge
418 deck with sandwich construction”, *Thin-Walled Structures*, 111: 138–144

419 Shan C (2017). “Bending performance of steel–polyurethane sandwich plate under local distributed load”, *Advances in*
420 *Structural Engineering*, 1–8.

421 Teixeira de Freitas, S., Kolstein, H., Bijlaard, F. (2010), “Composite bonded systems for renovations of orthotropic
422 steel bridge decks”, *Composite Structures*, **92**, 853–862.

423 Teixeira de Freitas, S., Kolstein, H., Bijlaard, F. (2011), “Sandwich system for renovation of orthotropic steel bridge
424 decks”, *Journal of Sandwich Structures and Materials*, **13**(3), 279–301.

425 Teixeira de Freitas, S., Kolstein, H., Bijlaard, F. (2012a), “Structural monitoring of a strengthened orthotropic steel
426 bridge deck using strain data”, *Structural Health Monitoring*, **11**(5), 558–376.

427 Teixeira de Freitas, S (2012b), *Steel plate reinforcement of orthotropic bridge deck*, PhD thesis, Delft University of
428 Technology, Netherlands (uuid:89af64f8-519b-4760-b717-515a3da4e27b).

429 Teixeira de Freitas, S., Kolstein, H., Bijlaard, F. (2013a), “Fatigue behavior of bonded and sandwich systems for
430 strengthening orthotropic steel bridge decks”, *Composite Structures*, **97**, 117–128.

431 Teixeira de Freitas, S., Kolstein, H., Bijlaard, F. (2013b), “Lightweight reinforcement systems for fatigue-cracked
432 orthotropic bridge decks”, *Structural Engineering International*, **4/2013**, 458-467.

433 Vincent R and Ferro A (2004). “A new orthotropic bridge deck: design, fabrication and construction of the Shenley
434 bridge incorporating a SPS orthotropic bridge deck”. In *2004 Orthotropic Bridge Conference*, Sacramento, USA.

435 Voermans J., Souren W. and Bosselaar M. (2014), “Strengthening the Orthotropic Steel Deck Structure of the
436 Movable Bridge across the Hartelkanaal”, *Structural Engineering International*, **3/2014**, 420-424.

437 Walter, R. (2005). *Cement-based overlay for orthotropic steel bridge decks: a multi-scale modeling approach*. PhD

438 thesis, Technical University of Denmark, Denmark.

439 Ya, S., Yamada, K., and Ishikawa, T. (2011), “Fatigue Evaluation of Rib-to-Deck Welded Joints of Orthotropic Steel

440 Bridge Deck”, *Journal of Bridge Engineering*, 16(4), 492-499.

441 Yamada, K., Ishikawa, T. and Kakiichi, T.(2015), “Rehabilitation and improvement of fatigue life of welded joints by

442 ICR treatment”, *Advanced Steel Construction*, 11(3), 294-304.

443 Zhang, S., Shao, X., Cao, J., Cui, J., Hu, J., and Deng, L (2016), “Fatigue Performance of a Lightweight Composite

444 Bridge Deck with Open Ribs”, *Journal of Bridge Engineering*, 21(7), 04016039.

445 Zhiyuan, Y., Bohai, J., Zhongqiu, F. and Hanbin G. (2016) “Fatigue Performance of Cracked Rib-Deck Welded Joint

446 Retrofitted by ICR Technique”, *International Journal of Steel Structures* 16(3): 735-742.

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451 Table 1: Maximum load (P_{max}) versus fatigue life (n_f) of the deck-plate-to-stiffener welds ($R = P_{min} / P_{max} = 0.1$).

Location	Bonded steel plates			Sandwich steel plates			
	P_{max} (kN)	n_f (cycles)		P_{max} (kN)	n_f (cycles)		
		10%	25%		10%	25%	
Crossbeam	160	72883	112354	160	139565	181320	
	160	63225	108994	160	62955	107528	
	160	64535	107649	160	52692	79330	
	160	48368	72561	160	46573	215155	
	110	402821	529840	110	125051	204498	
	110	65392	124920	110	274344	369033	
	110	107940	189030	110	126719	192044	
	110	120544	239029	110	291973	365773	
	90	227845	378262	90	125083	228472	
	90	98259	203896	90	145685	271185	
	90	211682	336889	90	125784	200292	
	90	116726	217144	90	>3918743 (run out)		
	Midspan	160	>5072367 (run out)		110	>4136051 (run out)	
		160	>5072367 (run out)		110	>4136051 (run out)	

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Table 2 – Retrofitting systems evaluated in the parametric study.

System	Nomenclature	Deck plate	Adhesive or Core ¹	2 nd steel plate	Weight
Bonded steel plates	B.12.2.6 ²	12 mm	2 mm	6 mm	49 kg/m ²
	B.12.2.8	12 mm	2 mm	8 mm	65 kg/m ²
	B.12.2.10	12 mm	2 mm	10 mm	81 kg/m ²
	B.12.2.12	12 mm	2 mm	12 mm	97 kg/m ²
Sandwich steel plates	S.12.15.5 ²	12 mm	15 mm	5 mm	57 kg/m ²
	S.12.20.5	12 mm	20 mm	5 mm	62 kg/m ²
	S.12.25.5	12 mm	25 mm	5 mm	68 kg/m ²
	S.12.30.5	12 mm	30 mm	5 mm	74 kg/m ²
	S.12.15.8	12 mm	15 mm	8 mm	80 kg/m ²
	B.12.30.6	12 mm	30 mm	6 mm	82 kg/m ²
	B.12.30.8	12 mm	30 mm	8 mm	97 kg/m ²

455 ¹ – *adhesive* when referred to bonded steel plates and *core* when referred to sandwich steel plates456 ² – retrofitting solution tested in the full-scale bridge decks

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459 Table 3 – Maximum shear stress τ_{xy} at the adhesive layer and at the steel-core of the bonded and sandwich steel plates system

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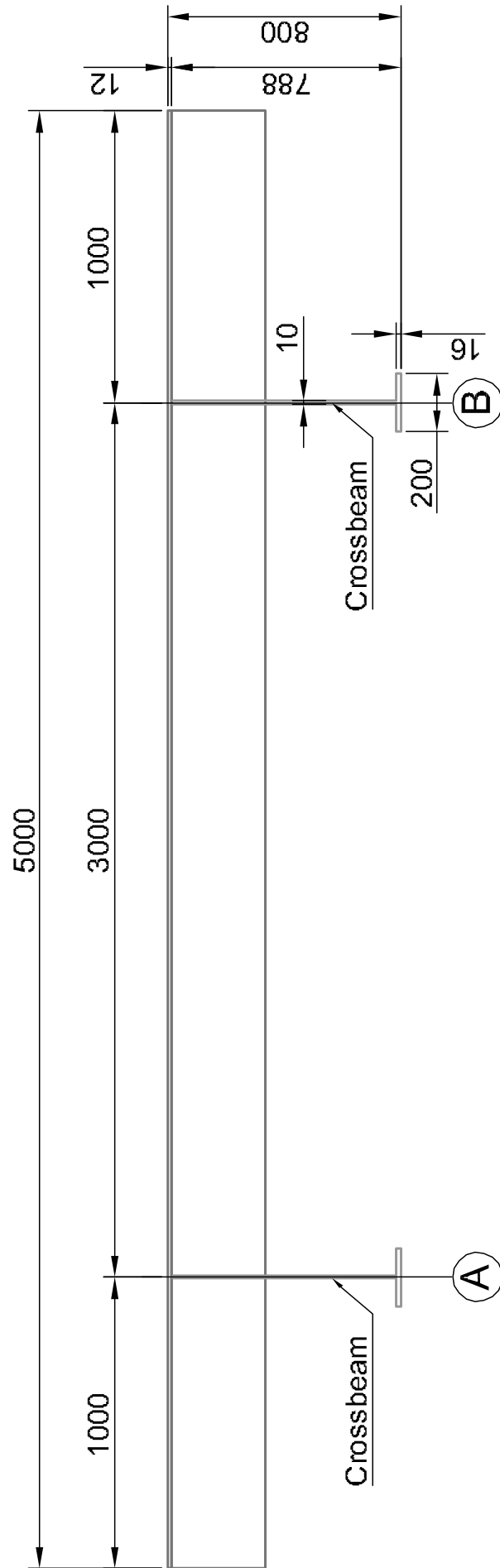
(100 kN load, wheel type C).

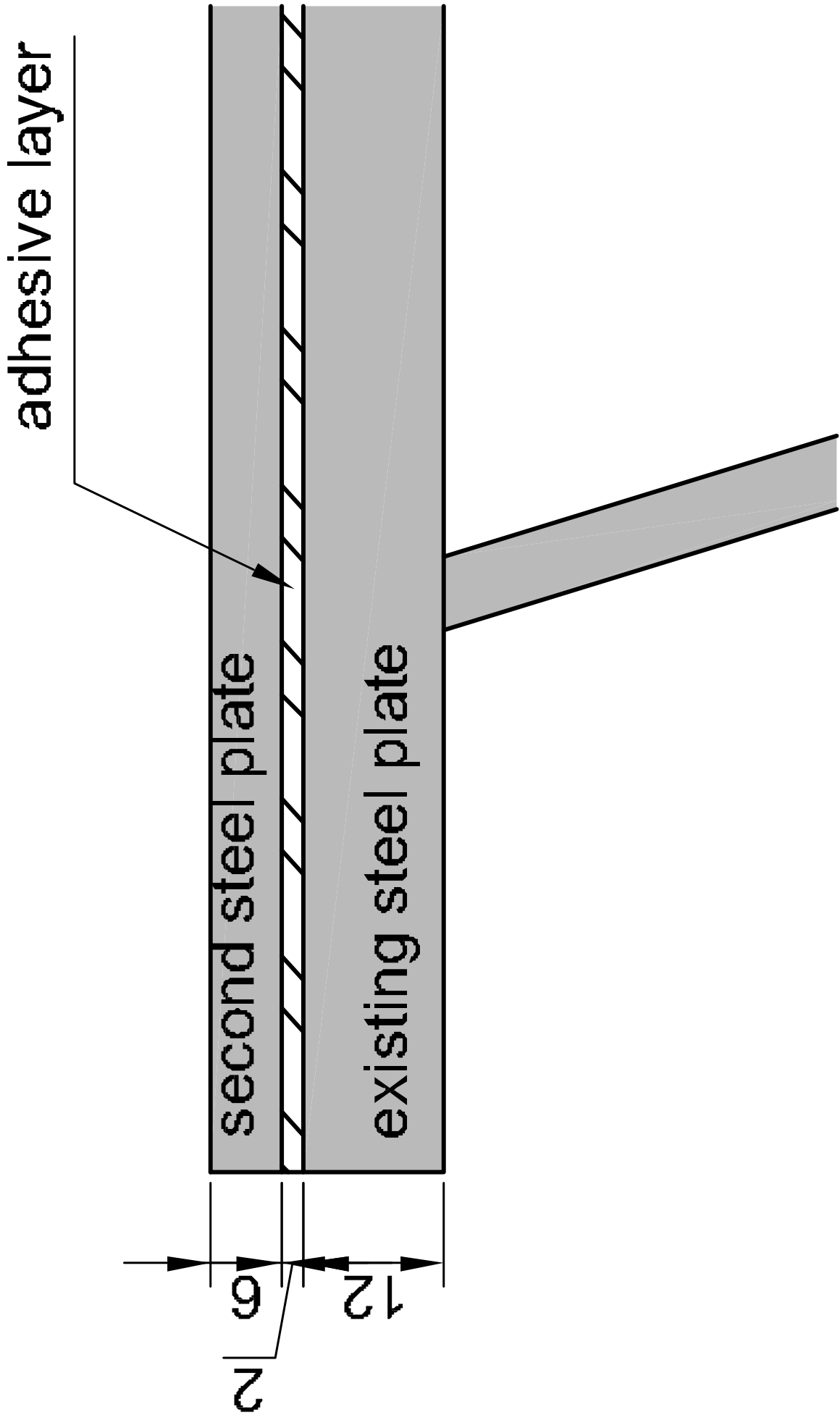
τ_{xy} (MPa)	Nomenclature	Adhesive or Interface steel-core ¹	
		Crossbeam location	Midspan between crossbeams
Bonded steel plates	B.12.2.6 ²	7.95	6.95
	B.12.2.8	7.63	6.66
	B.12.2.10	7.14	6.18
	B.12.2.12	6.61	5.64
Sandwich steel plates	S.12.15.5 ²	2.35	2.18
	S.12.20.5	2.23	1.97
	S.12.25.5	2.06	1.92
	S.12.30.5	1.88	1.71
	S.12.15.8	1.74	1.59
	B.12.30.6	1.69	1.55
	B.12.30.8	1.57	1.43

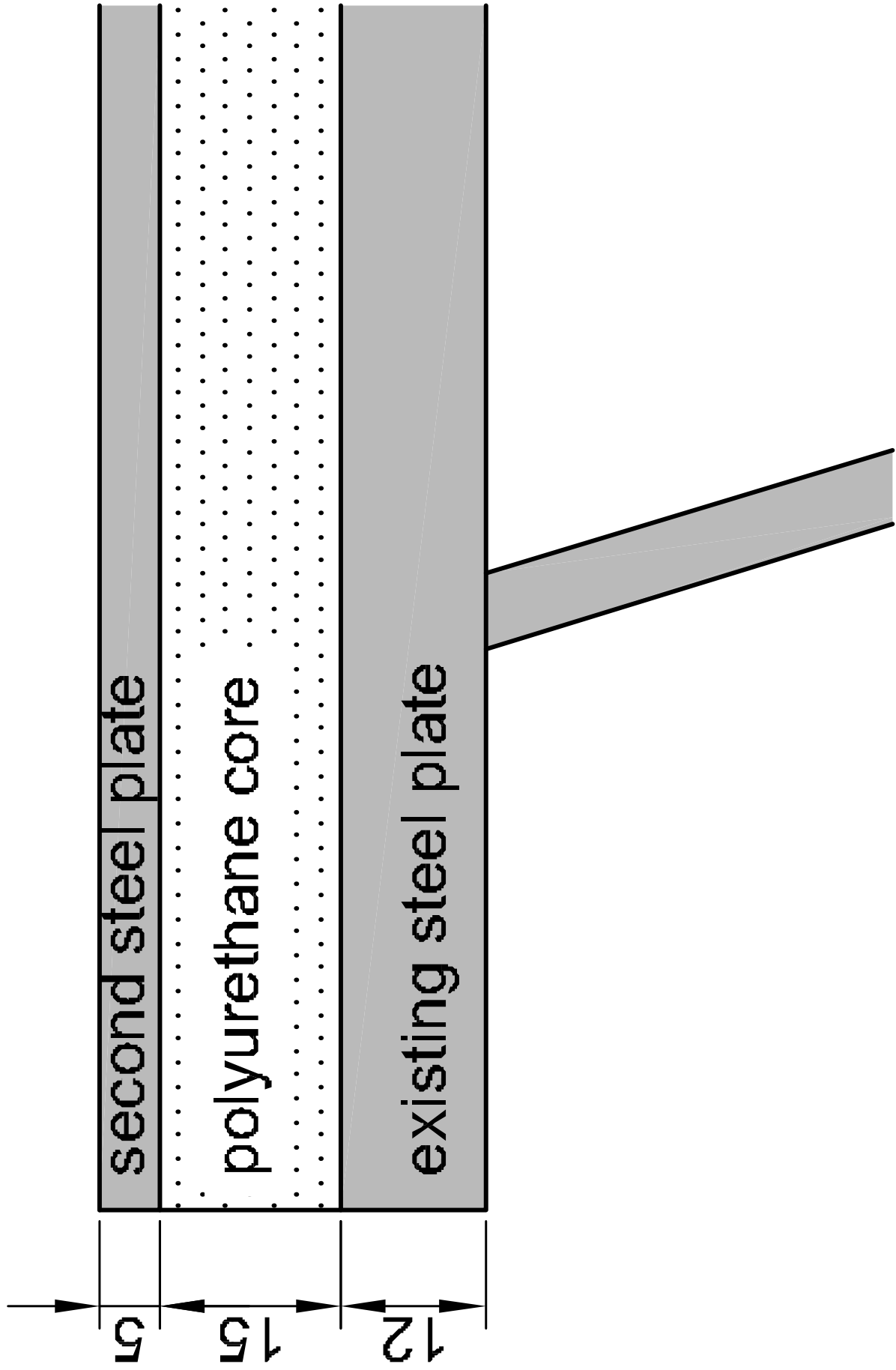
461 ¹ – *Adhesive* when referred to bonded steel plates and *Interface steel-core* when referred to sandwich steel plates462 ² – retrofitting solution tested in the full-scale bridge decks

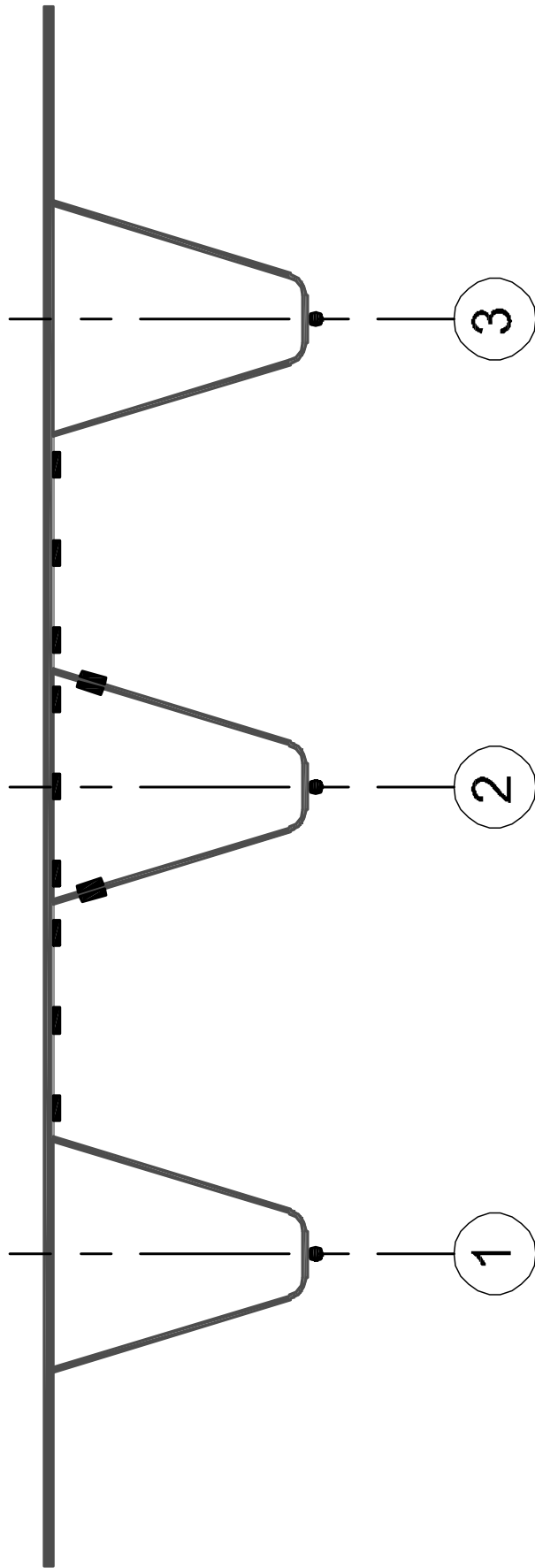
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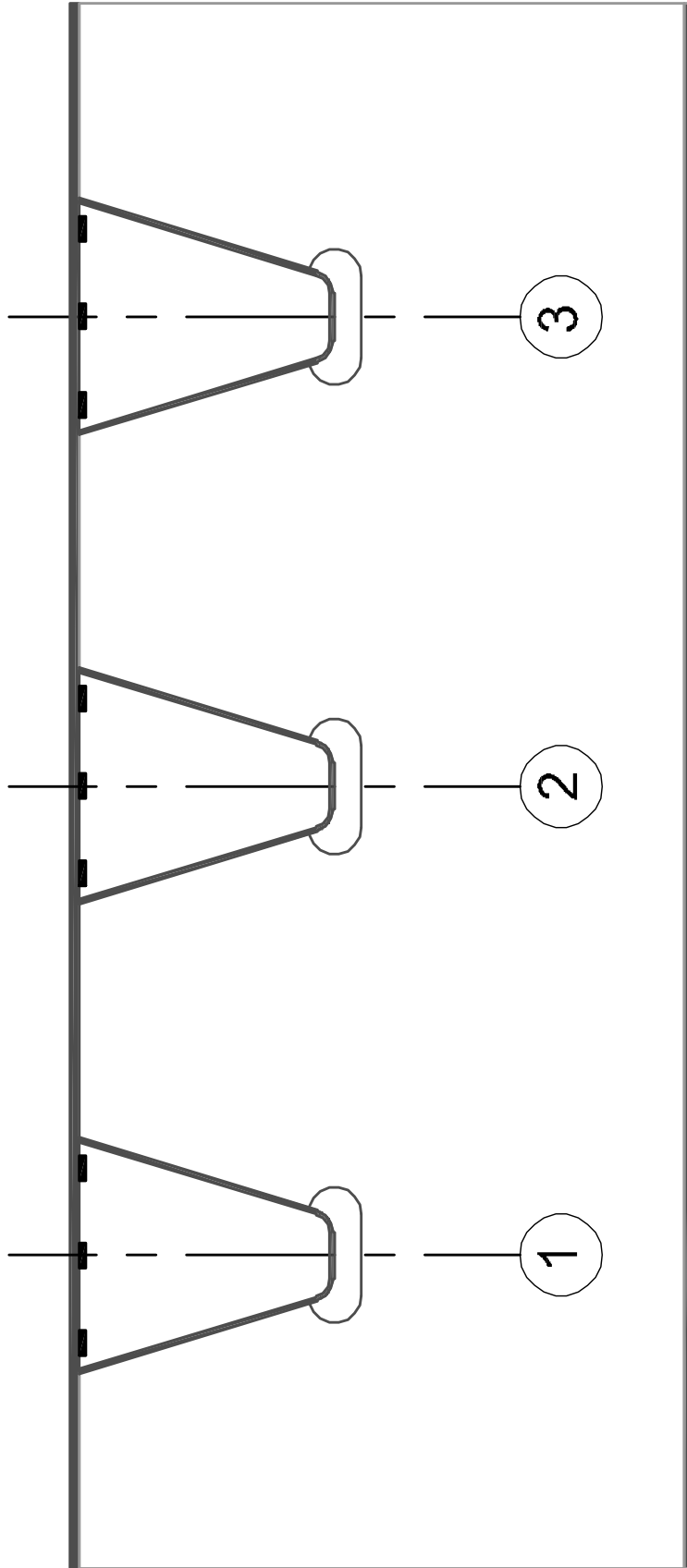
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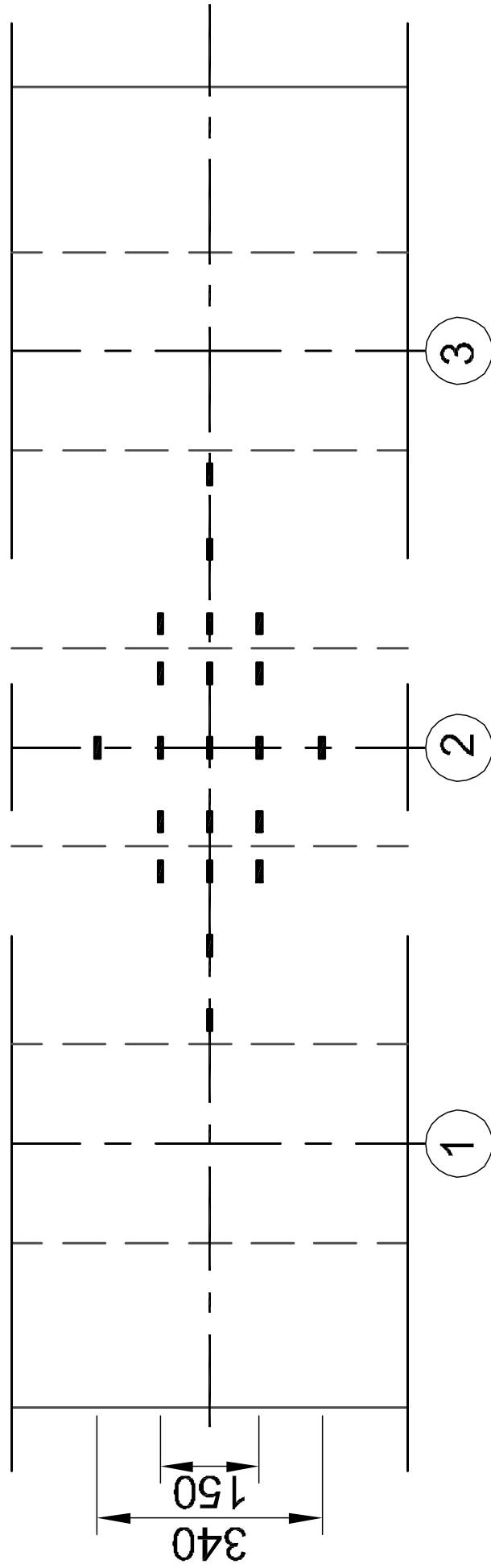


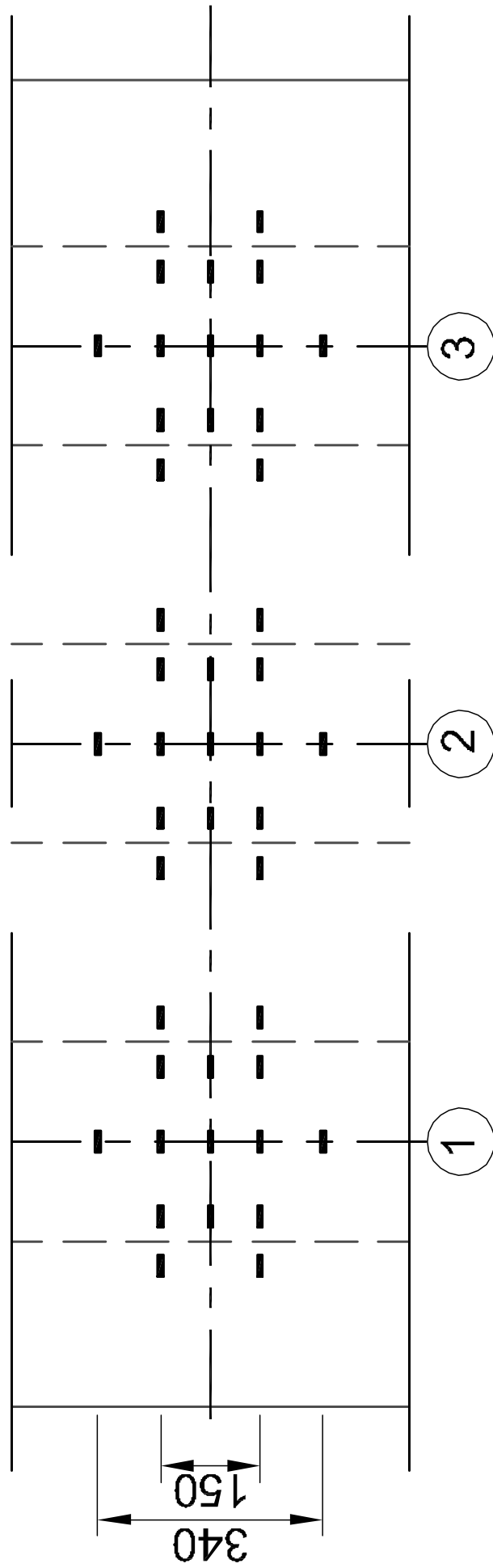


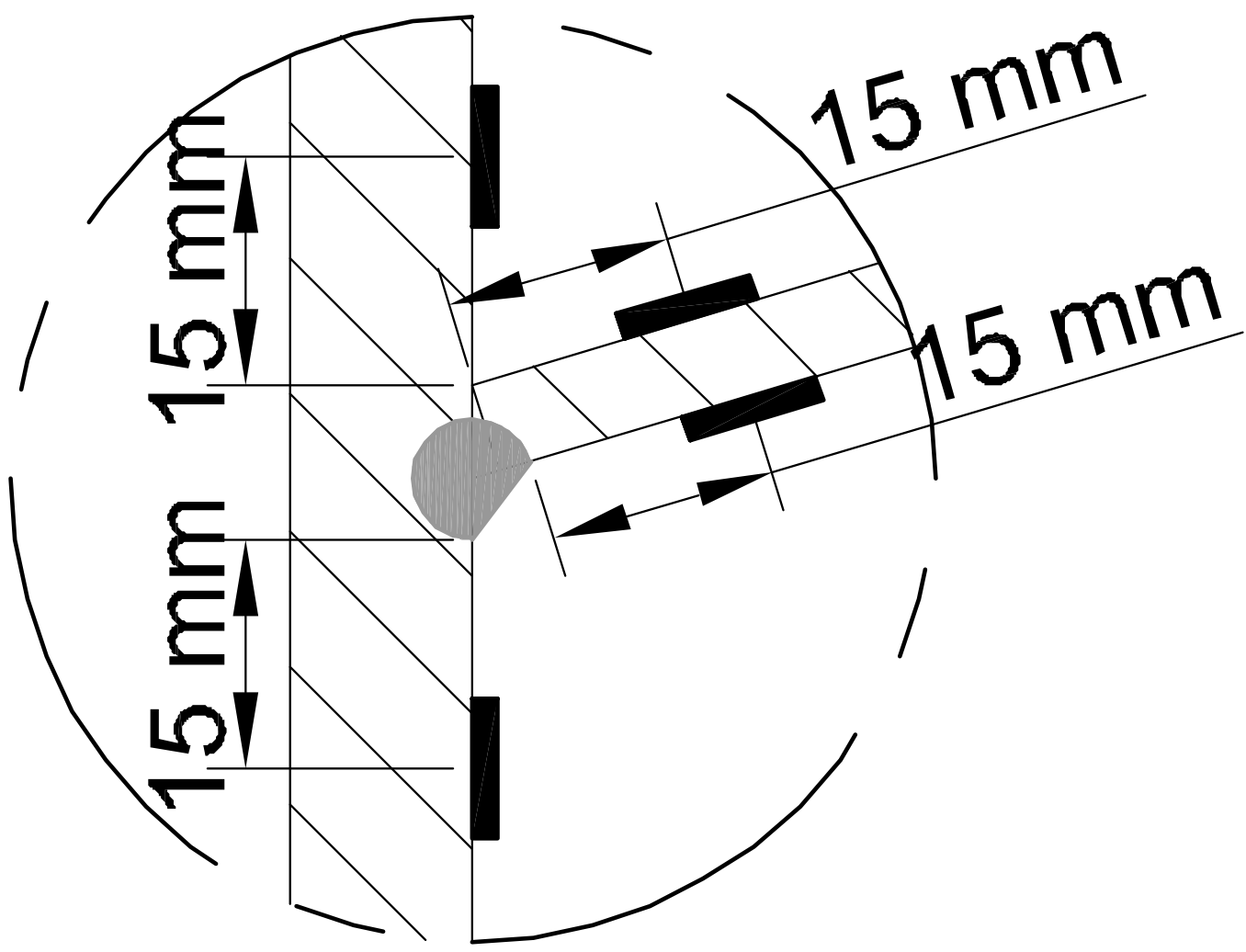


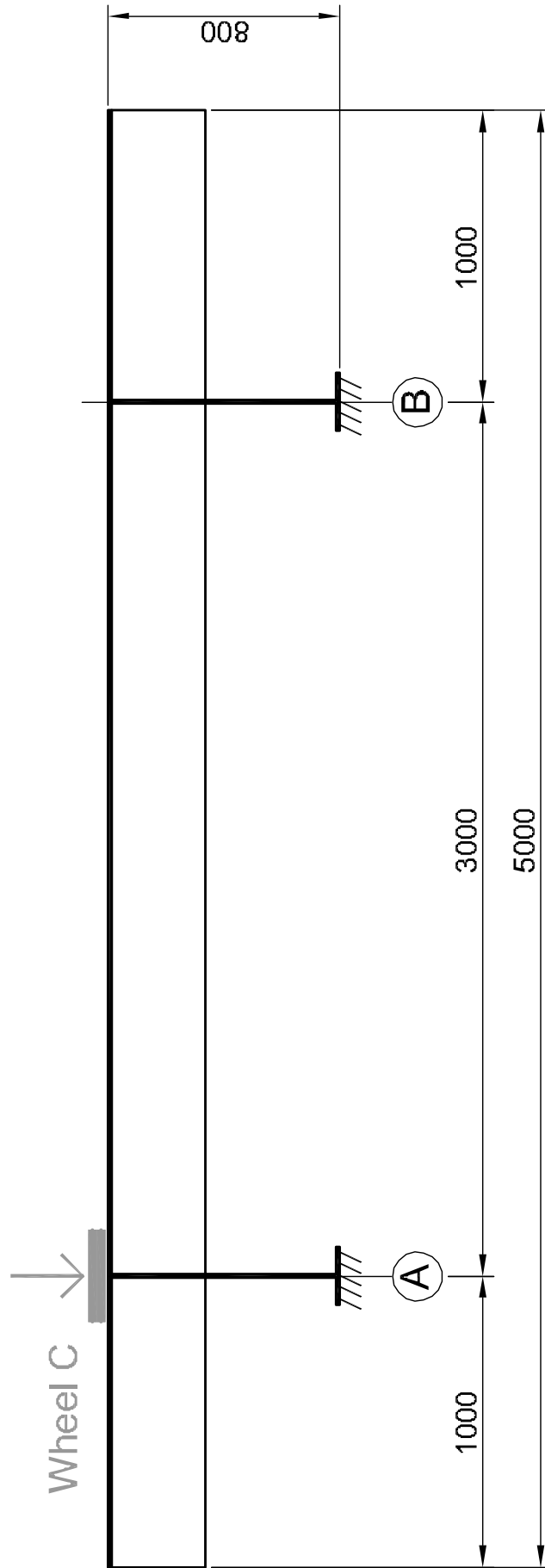












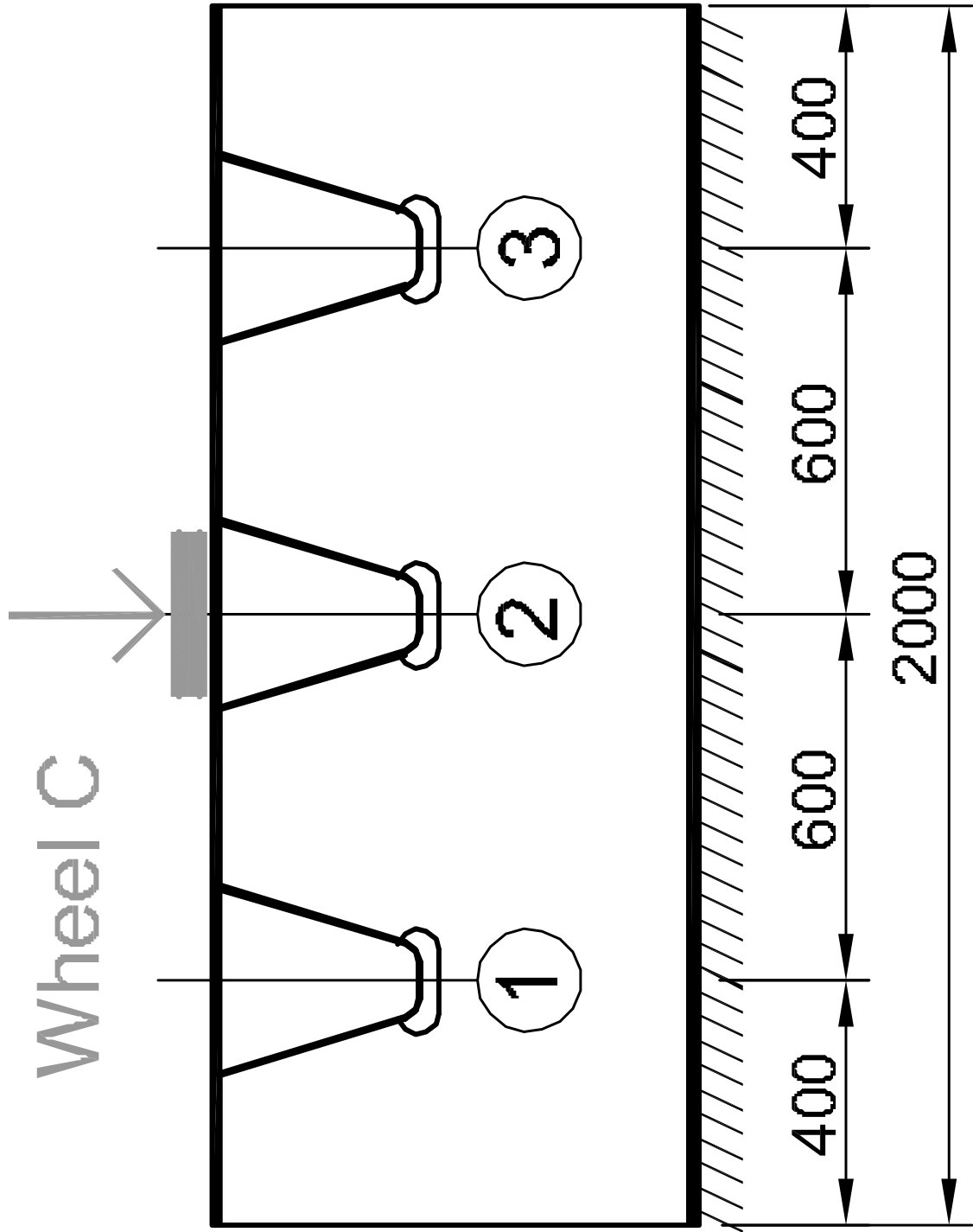
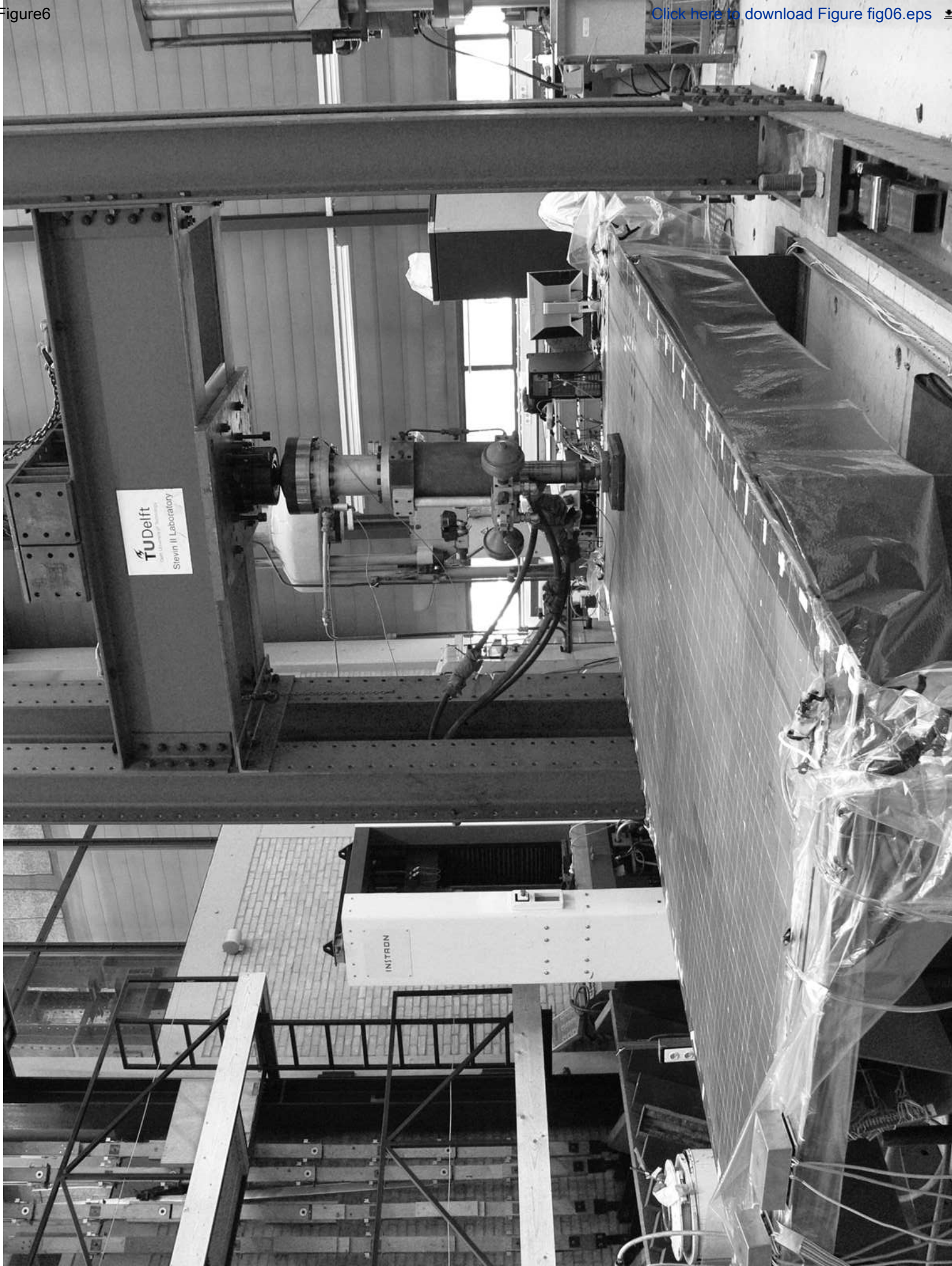
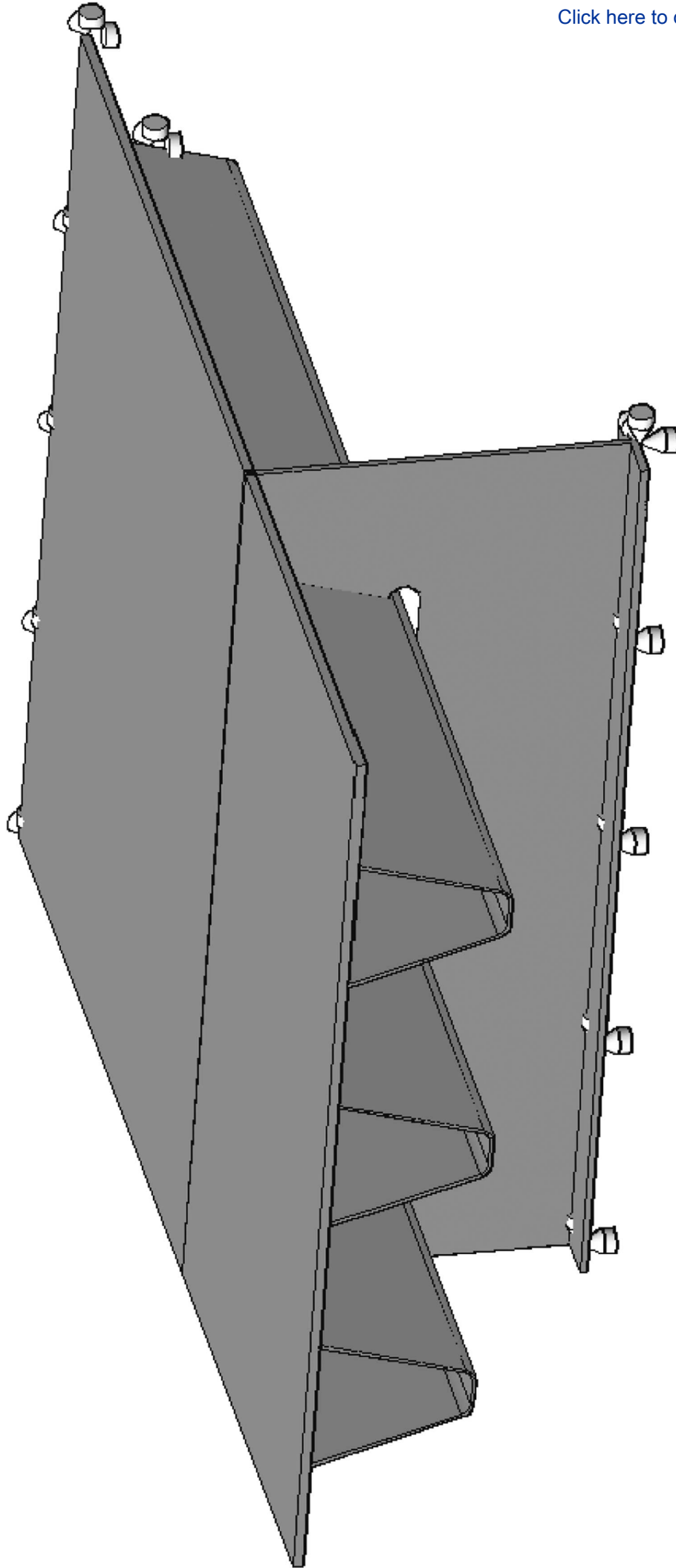


Figure6





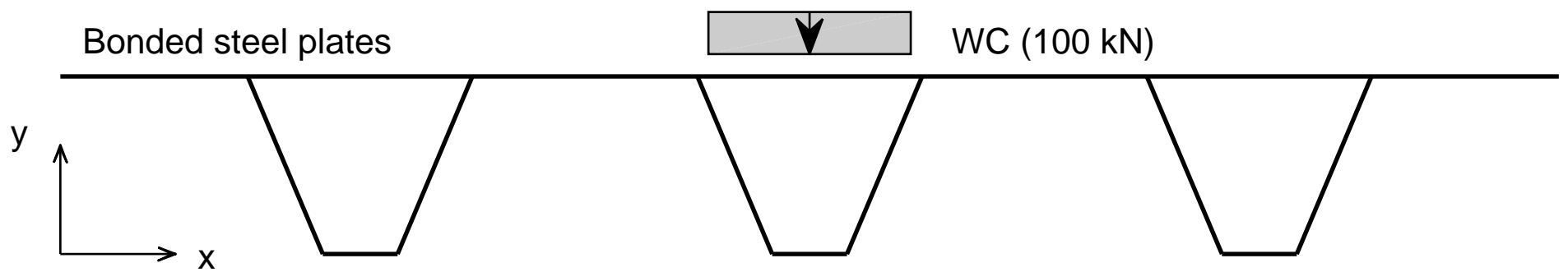
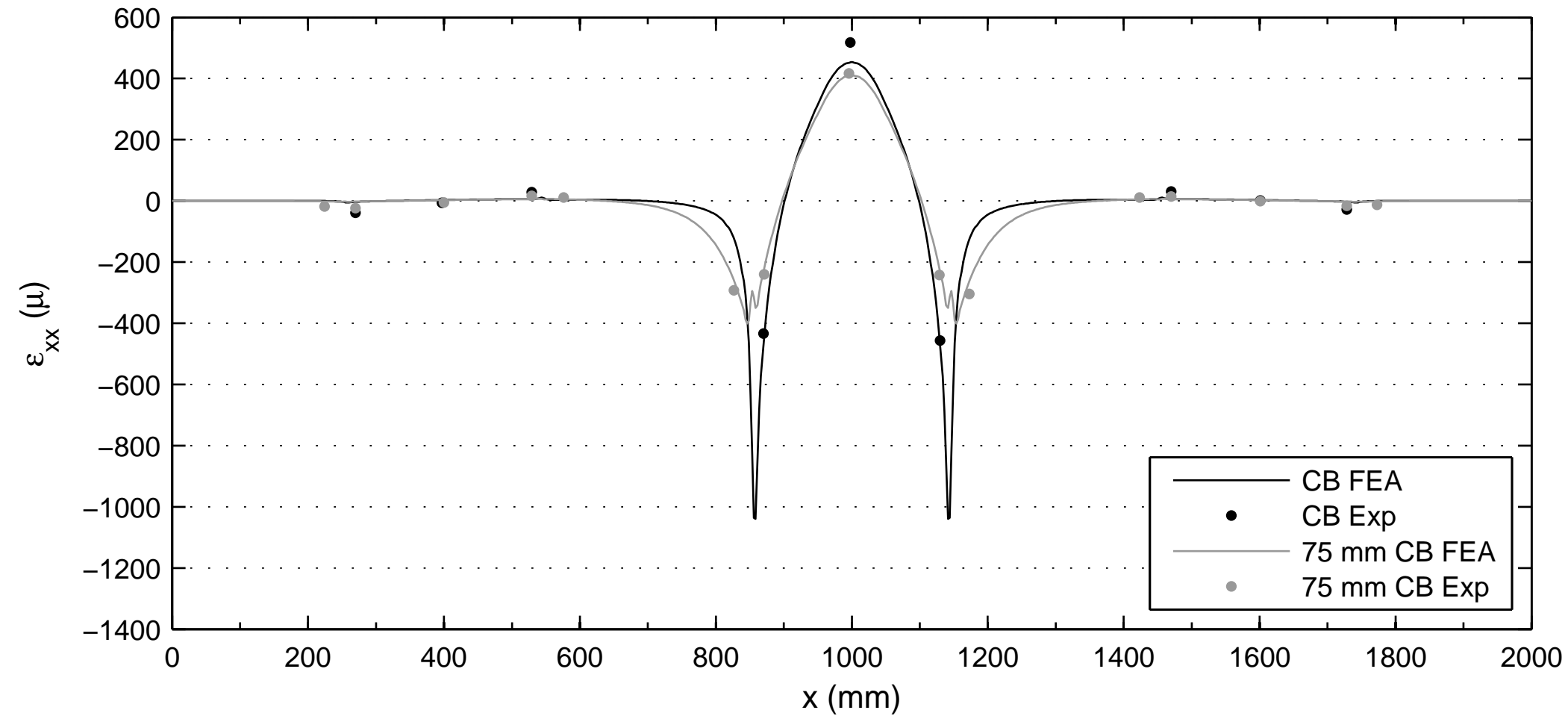
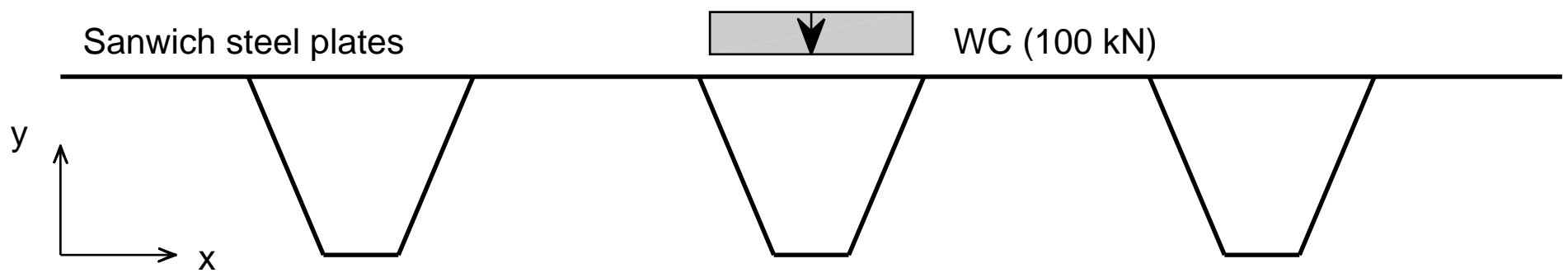
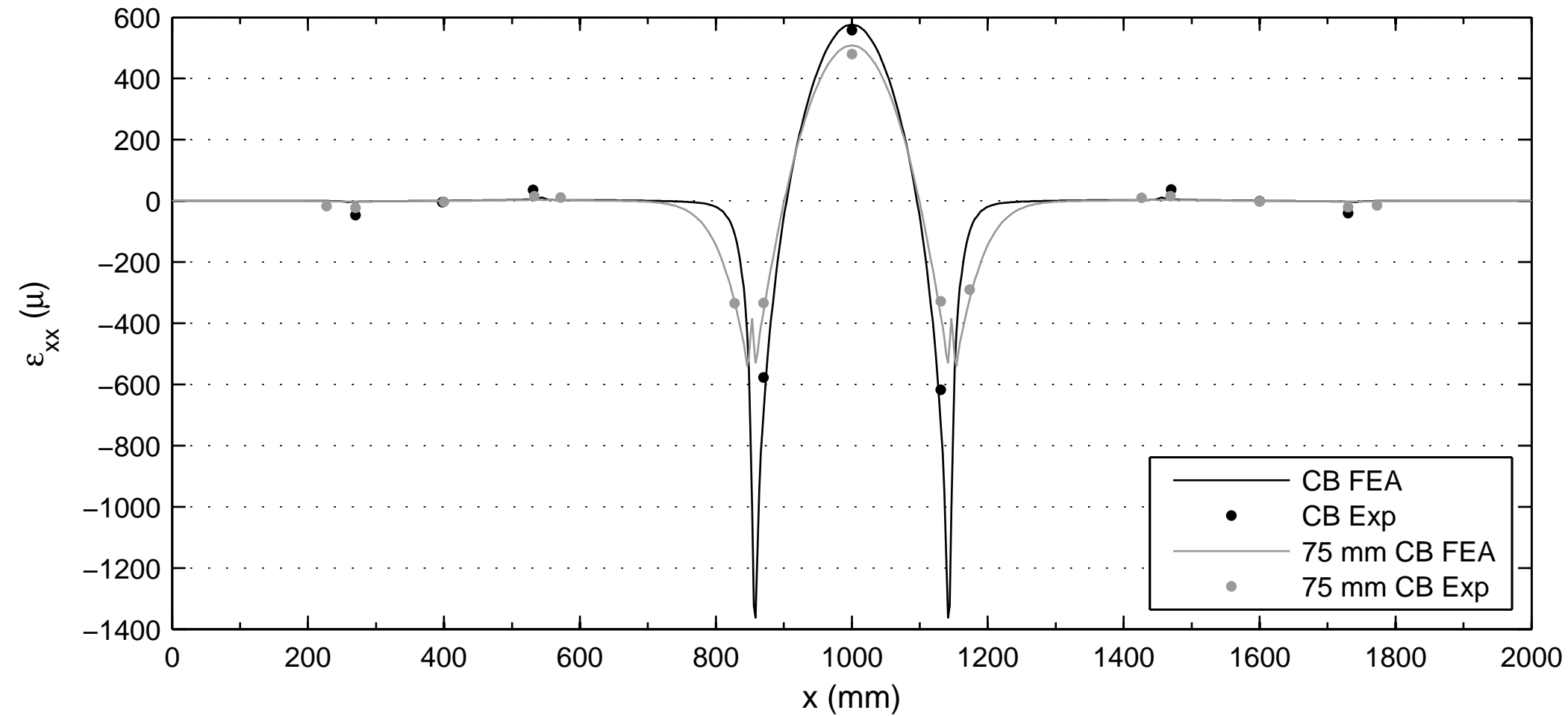
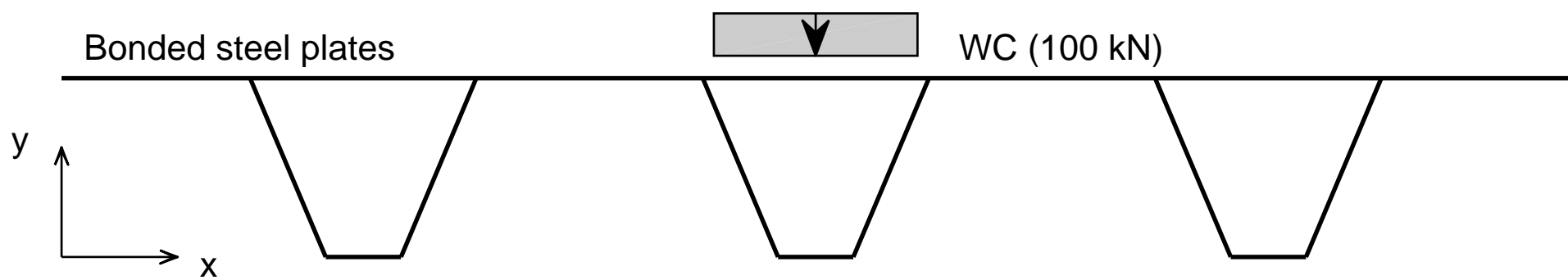
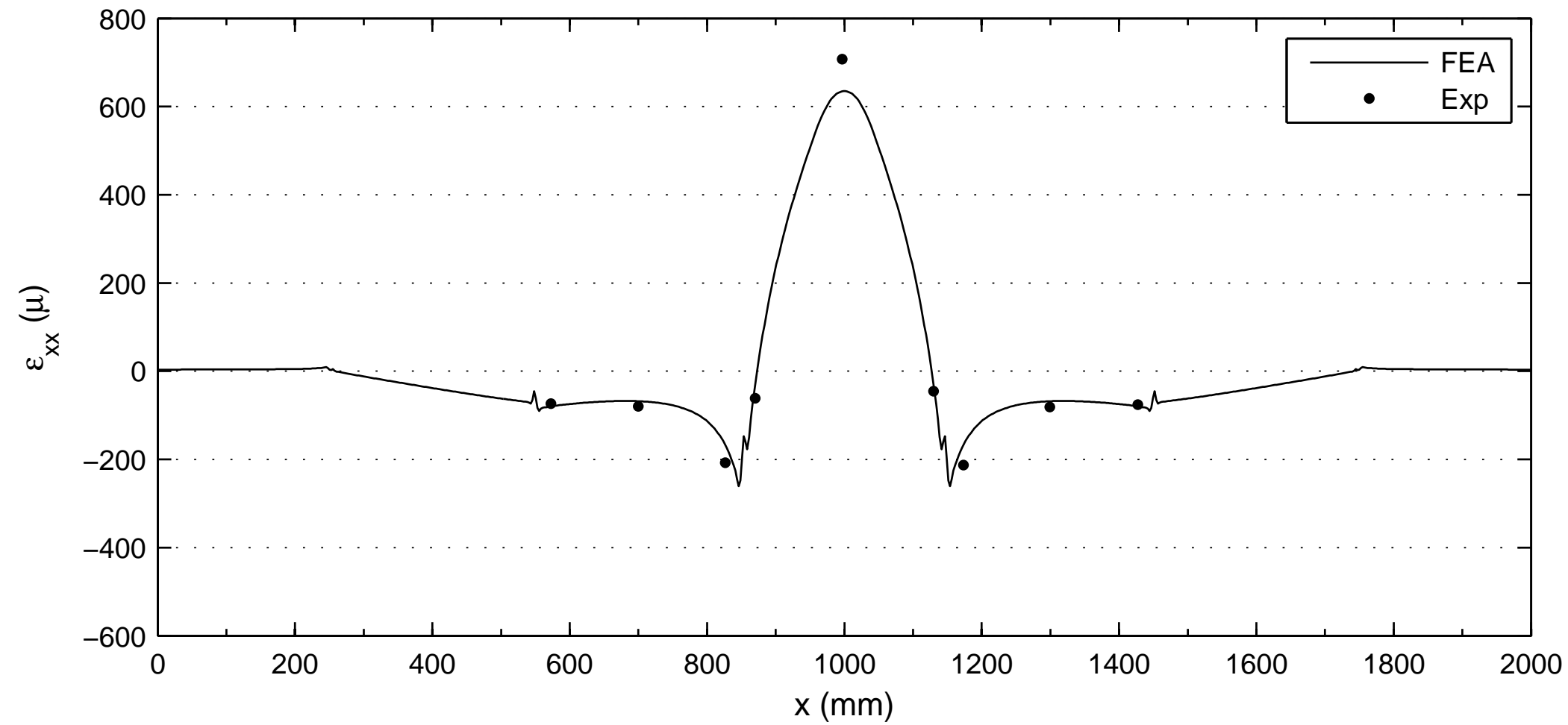
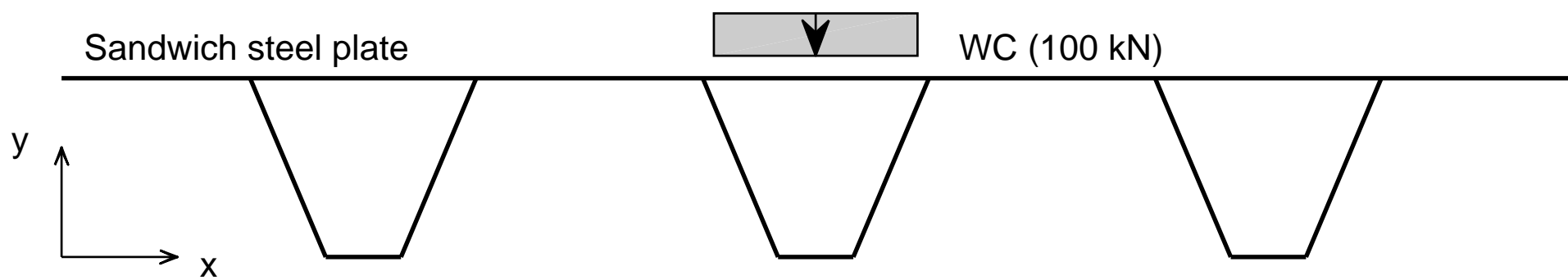
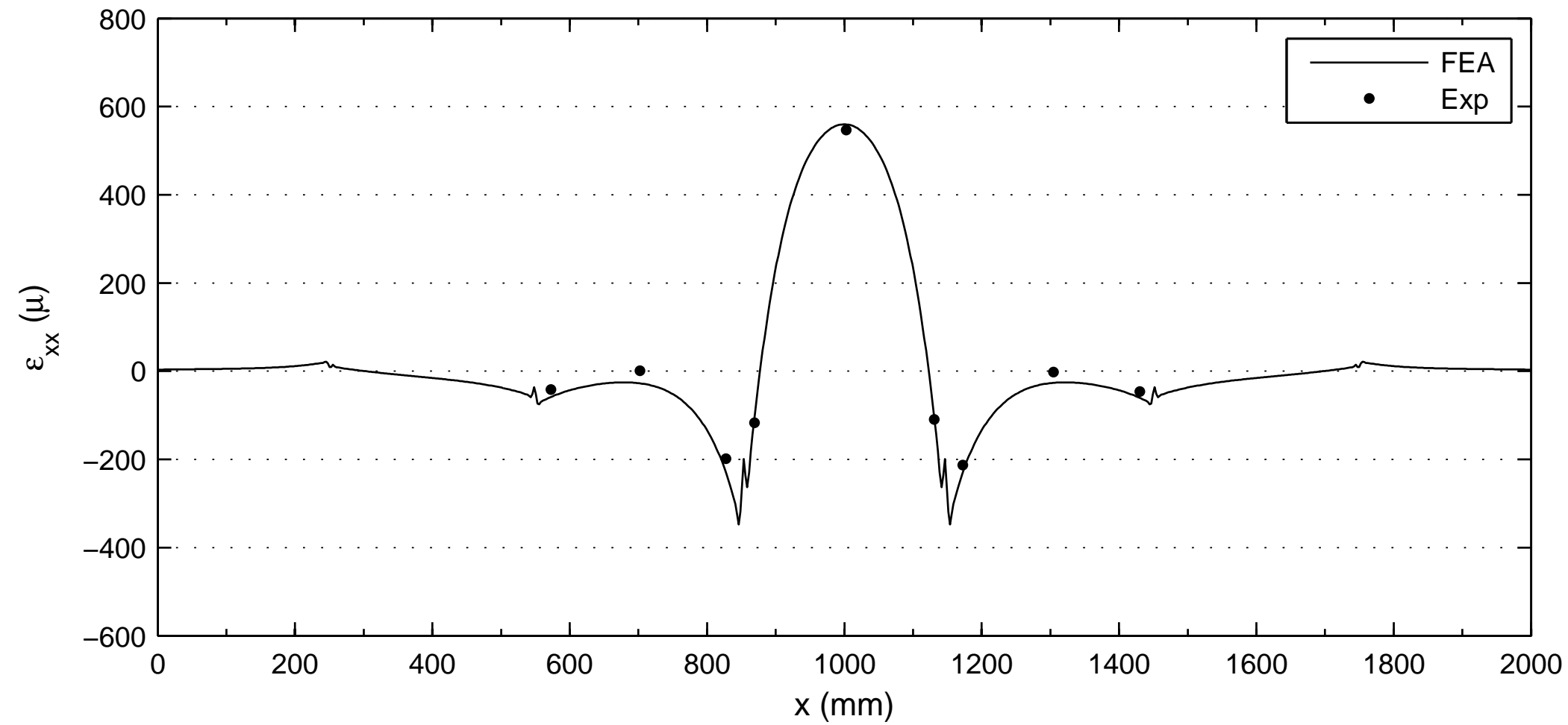
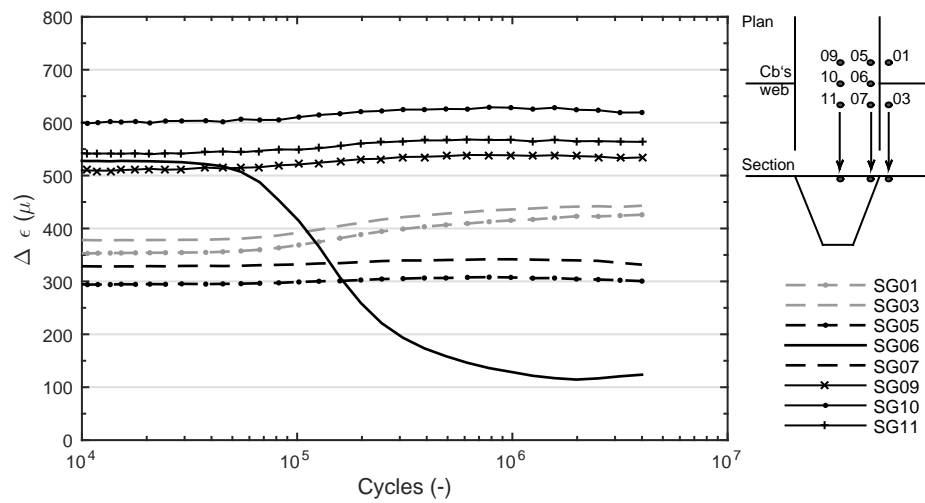


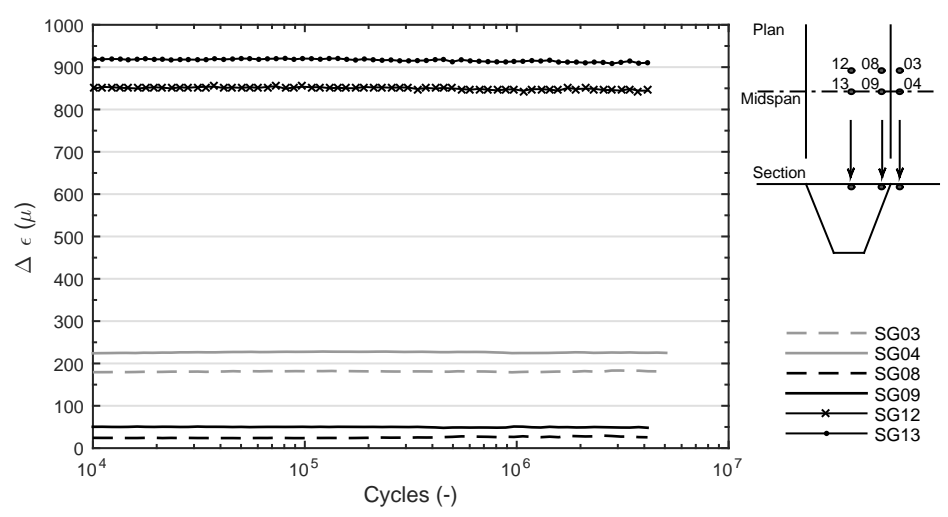
Figure8b

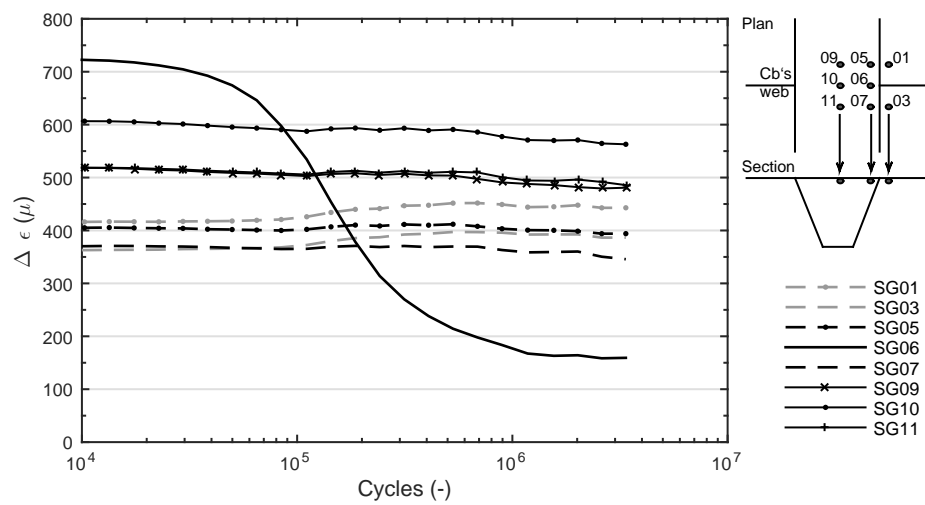


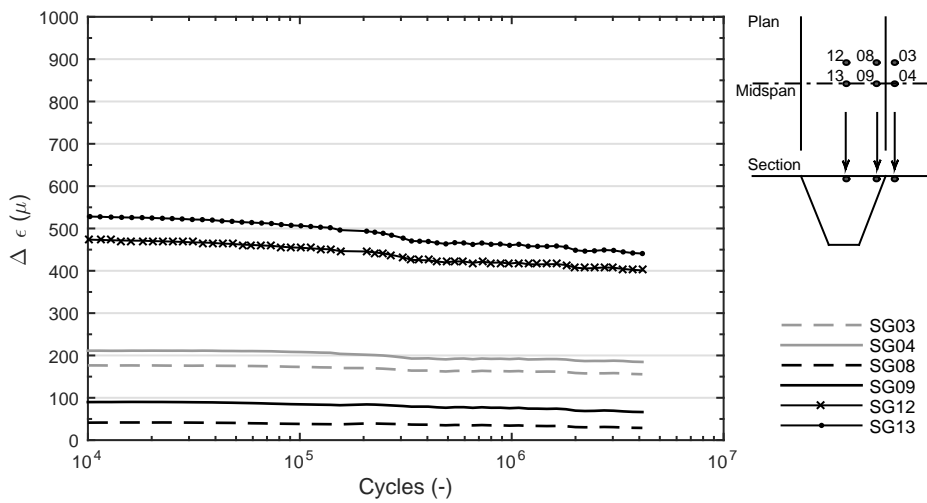


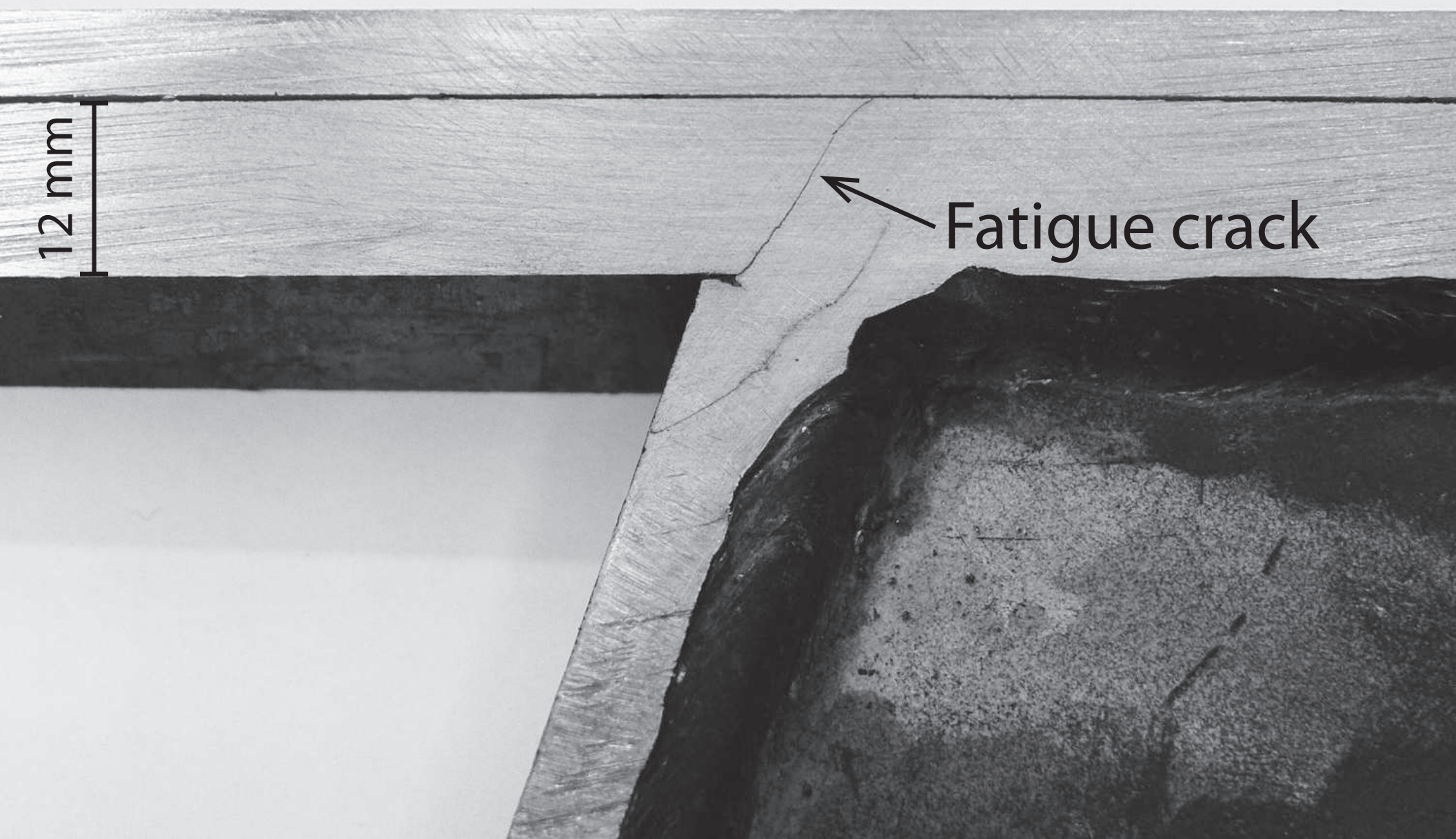








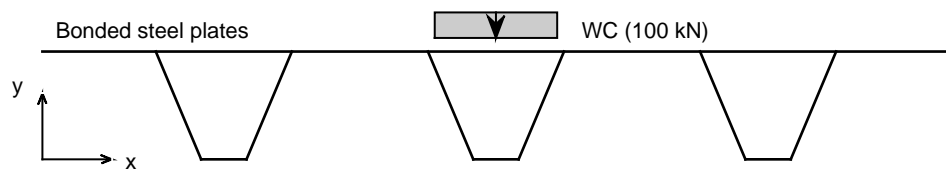
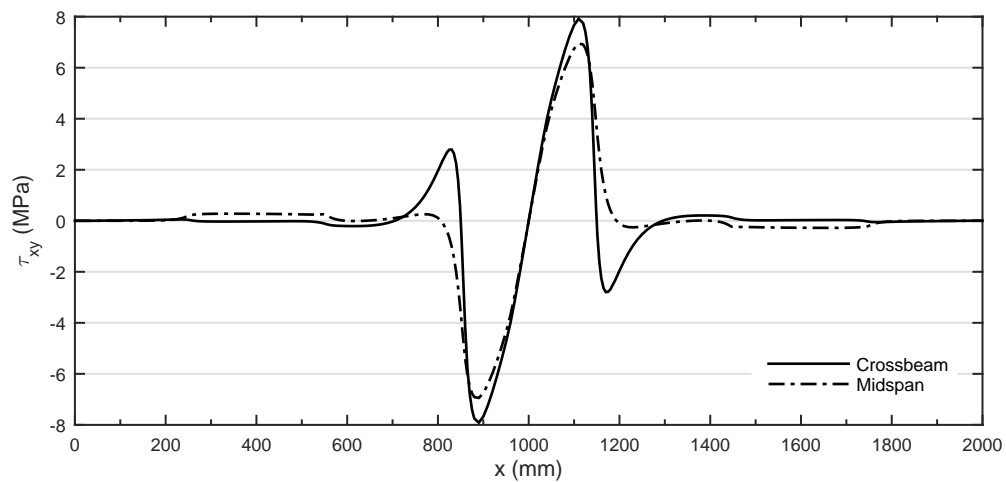


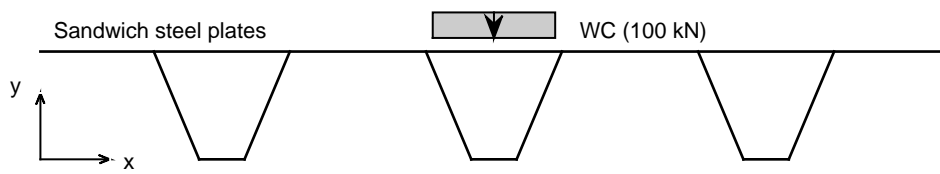
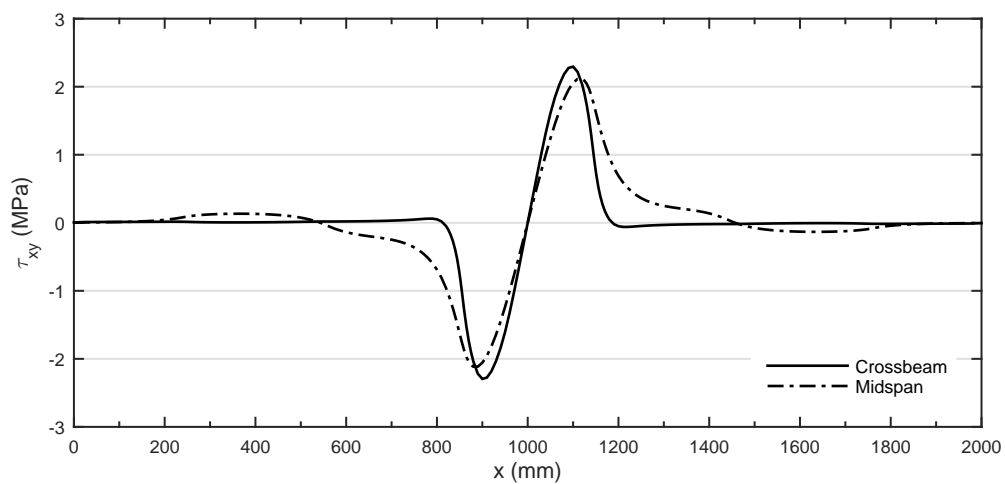


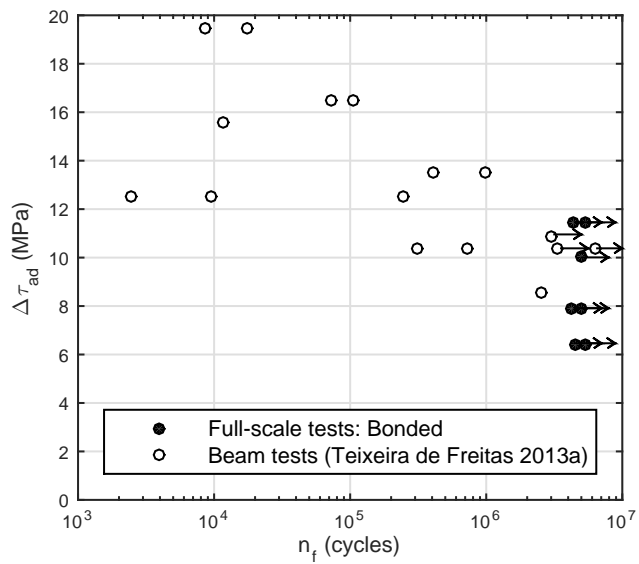


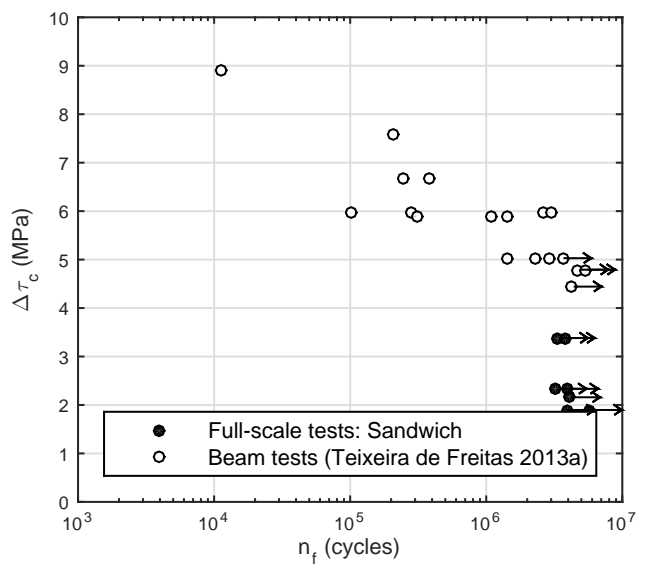
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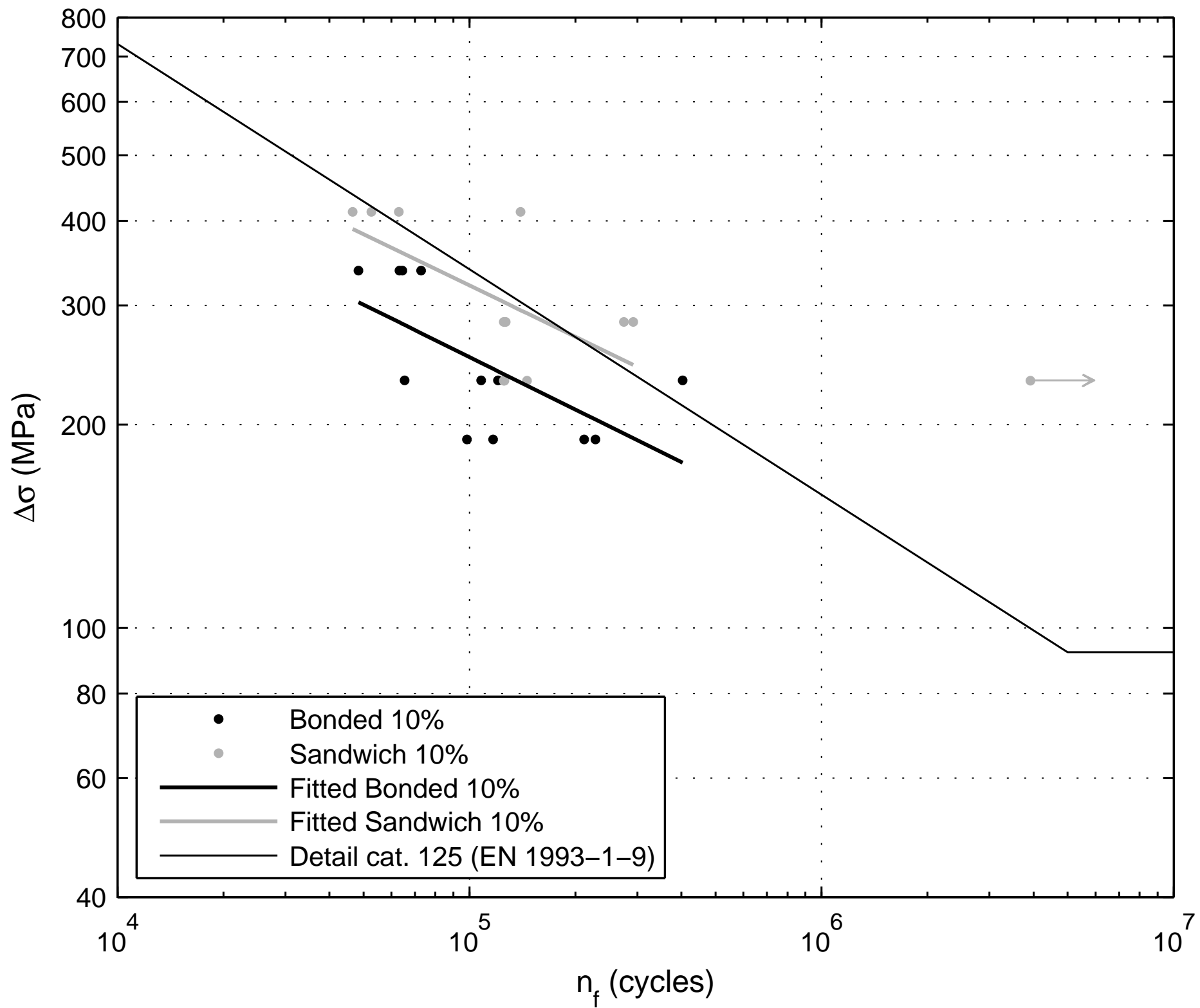
Fatigue crack











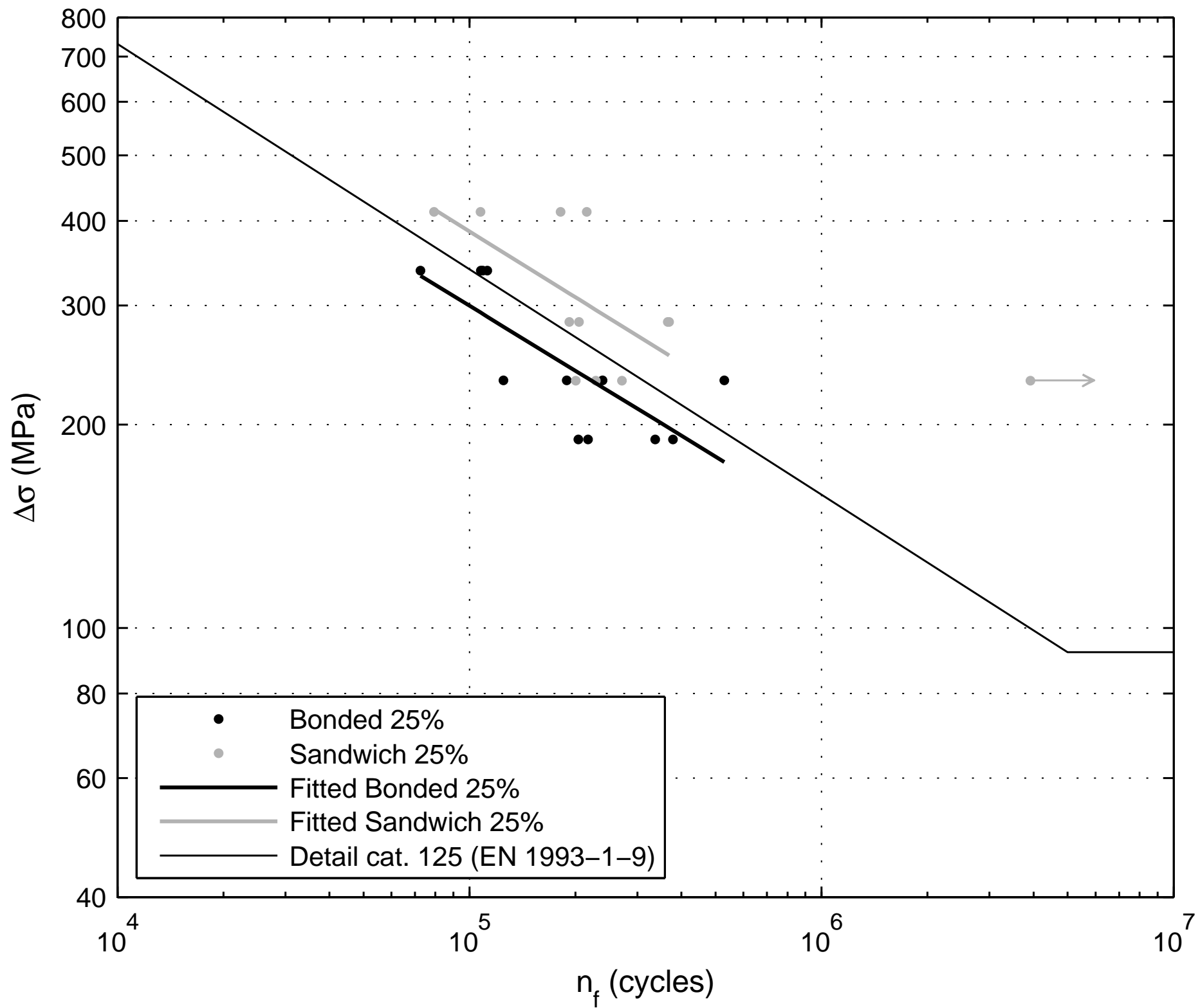
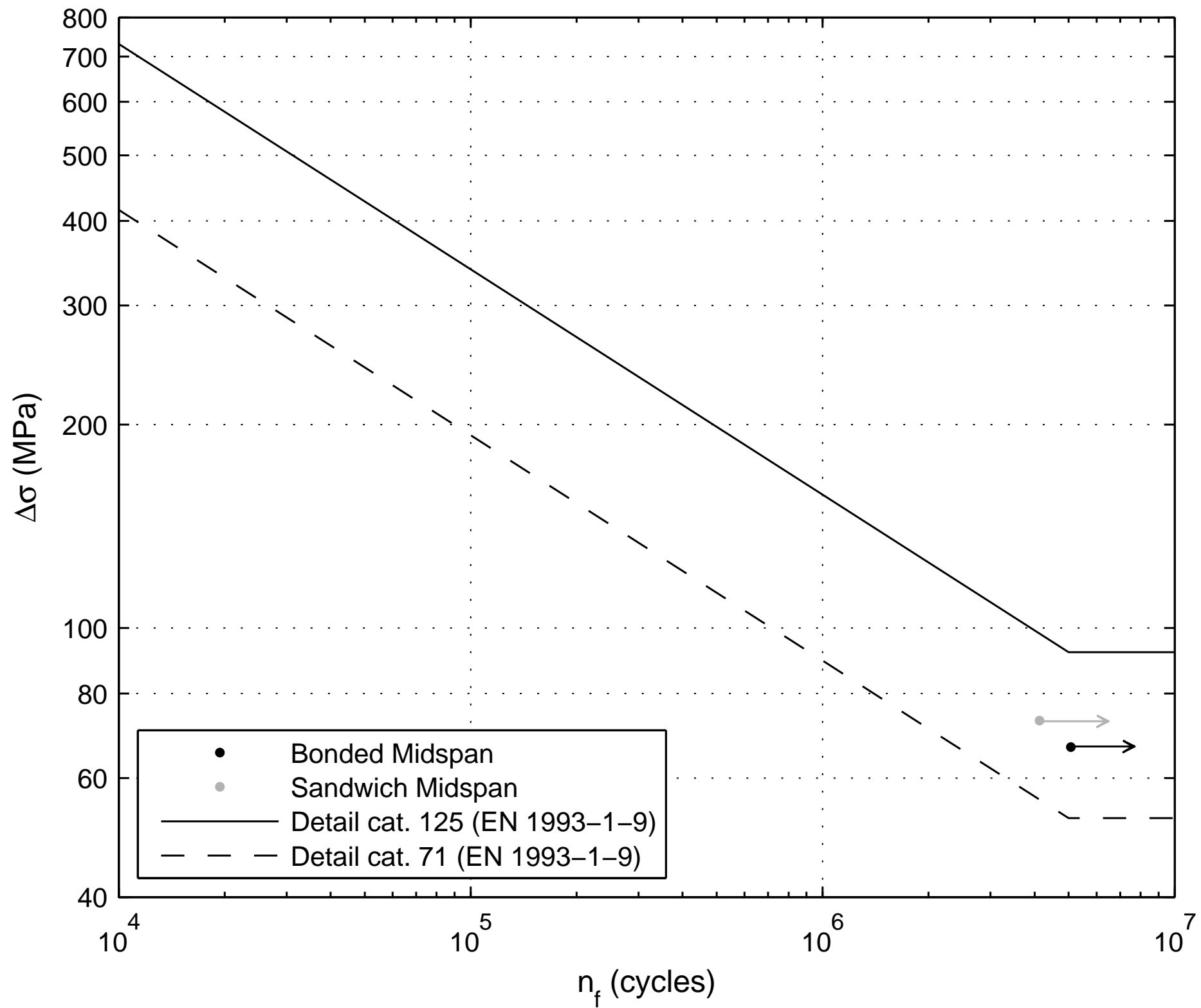
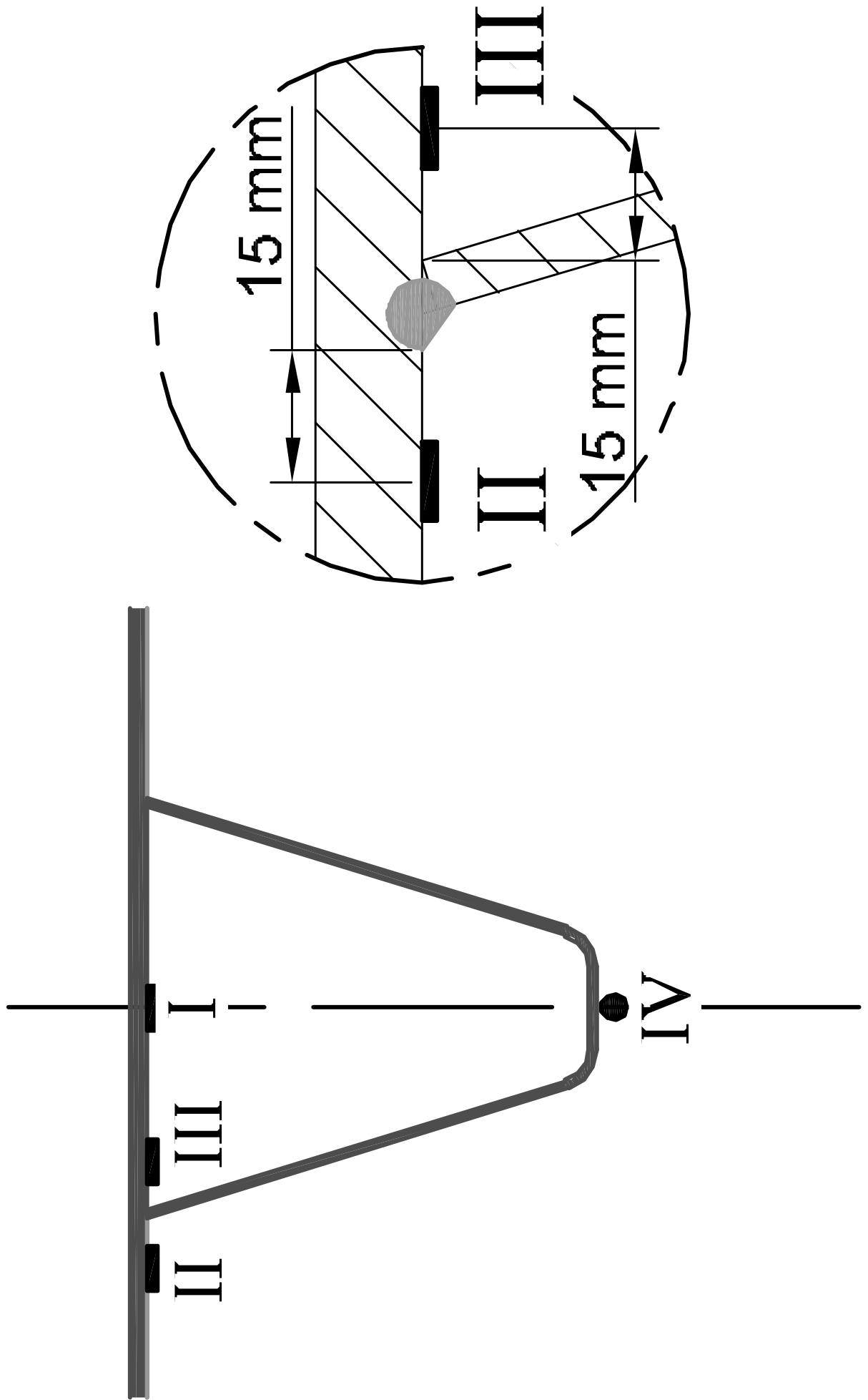
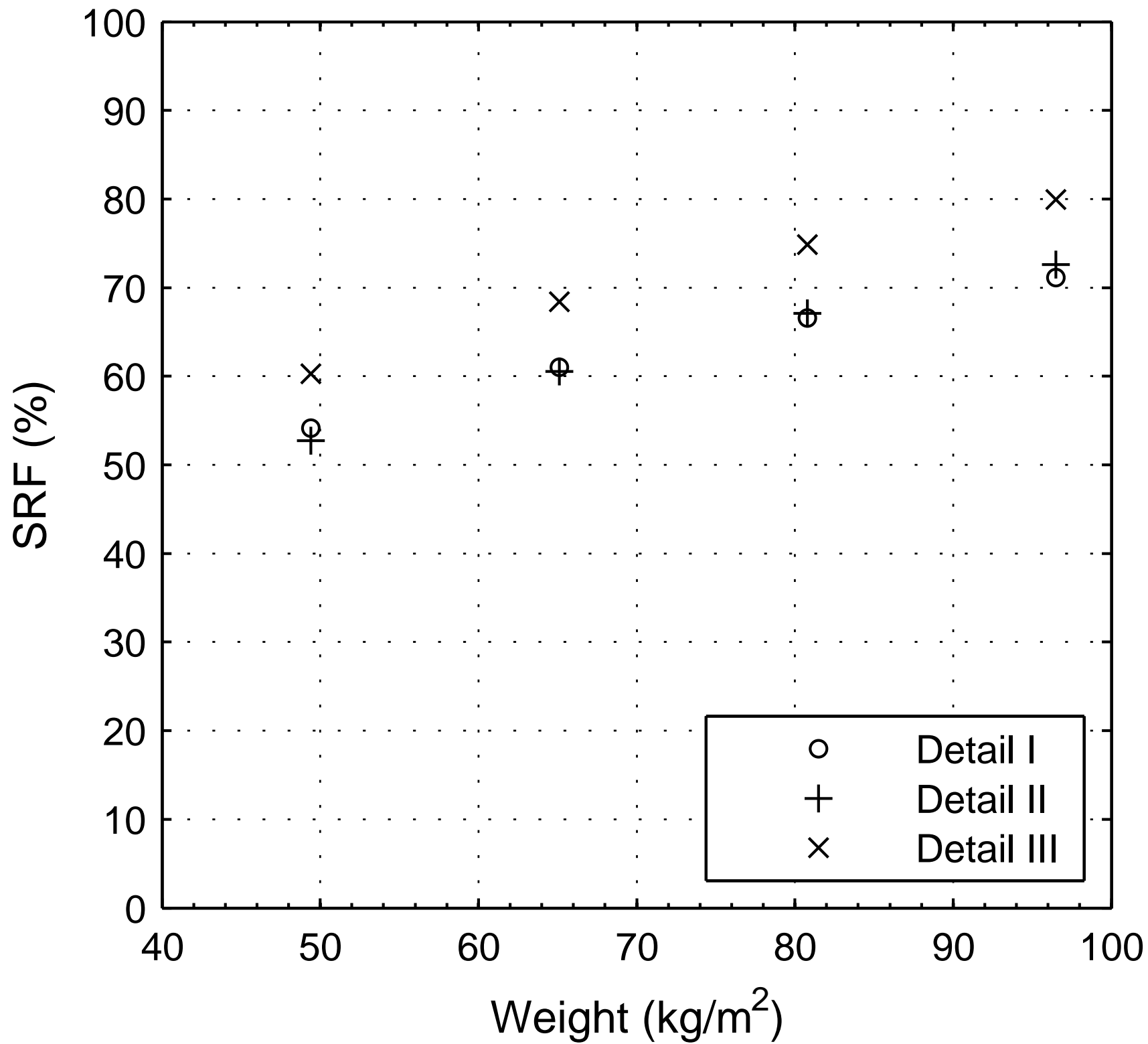
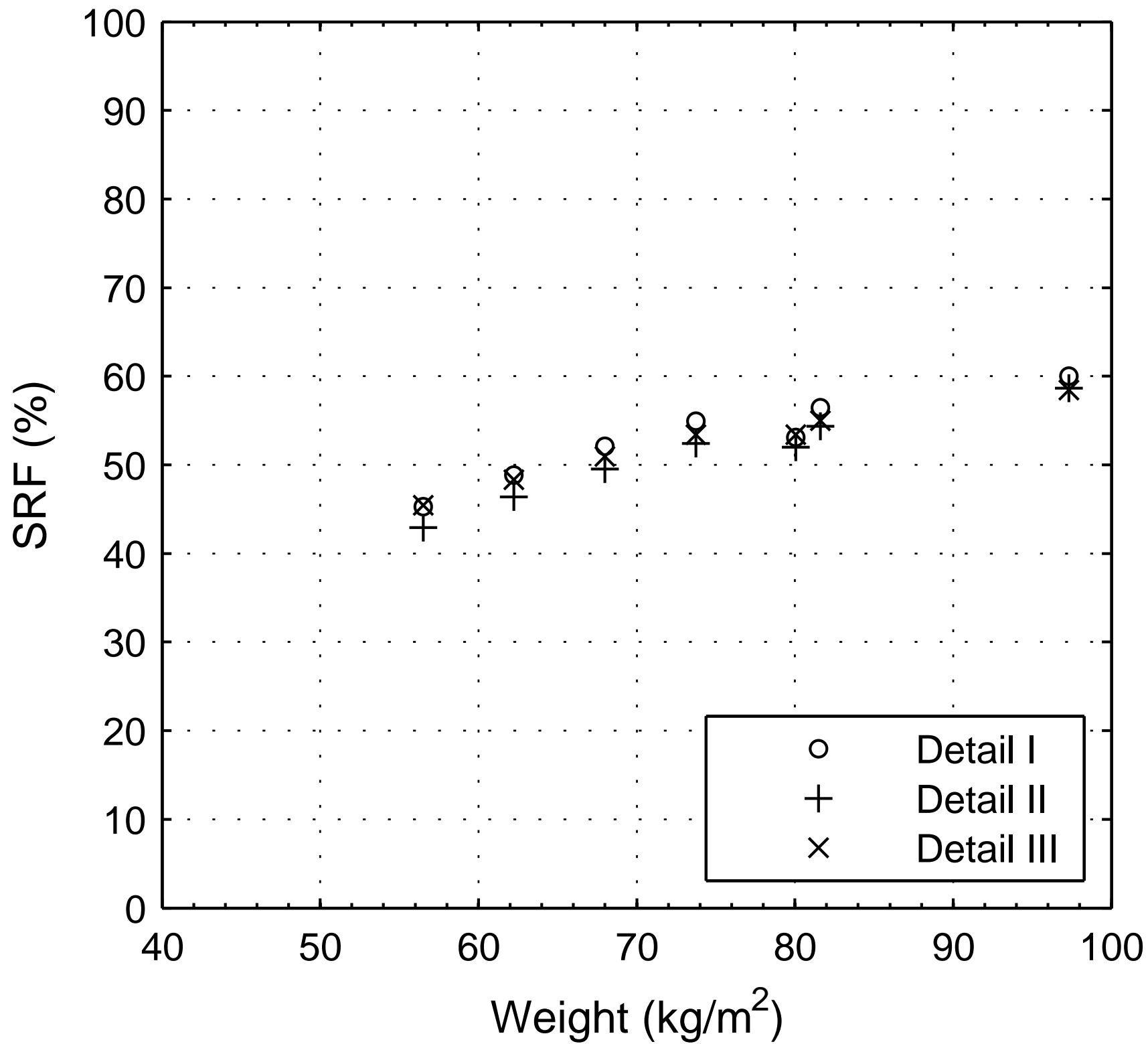


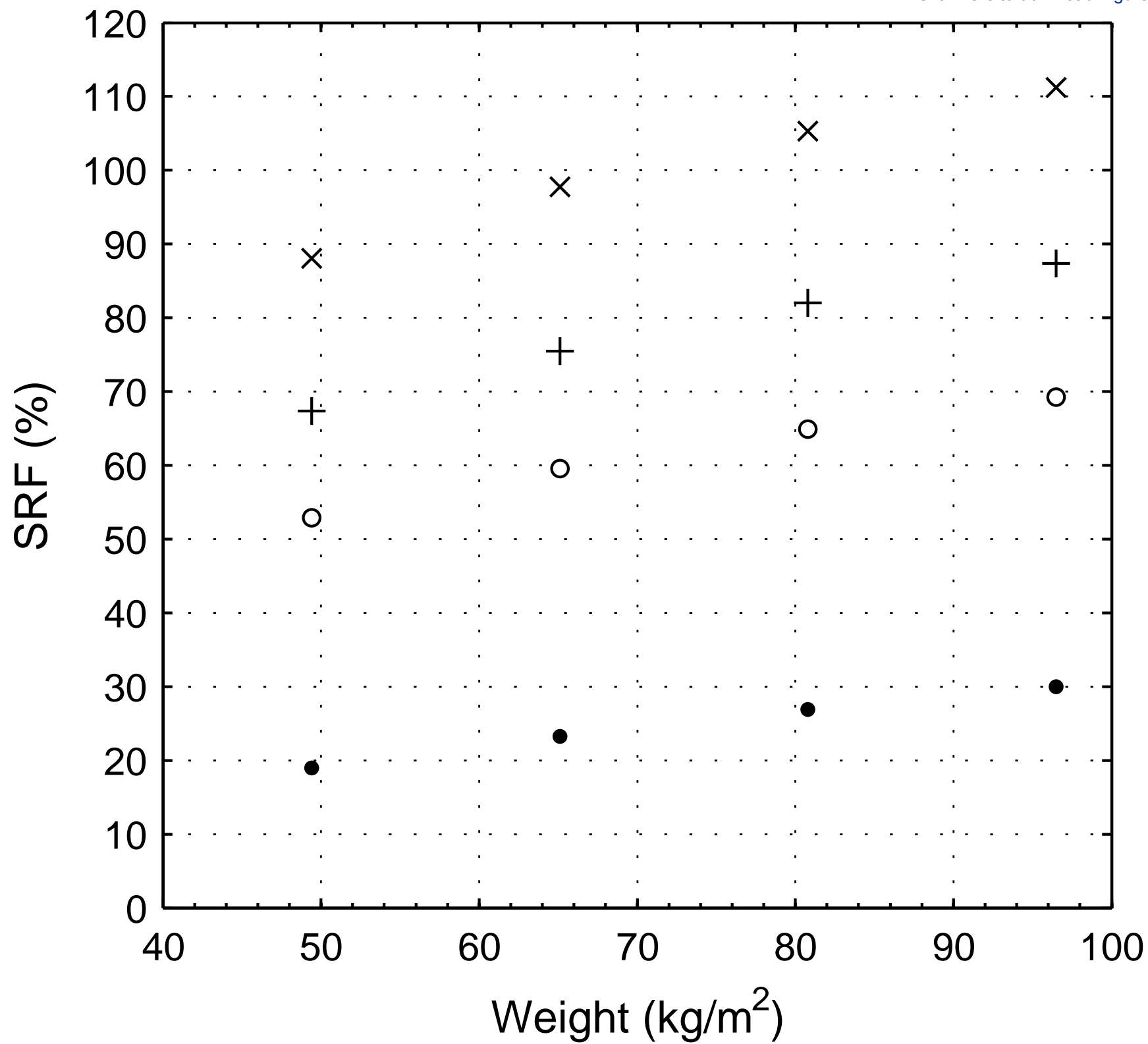
Figure16











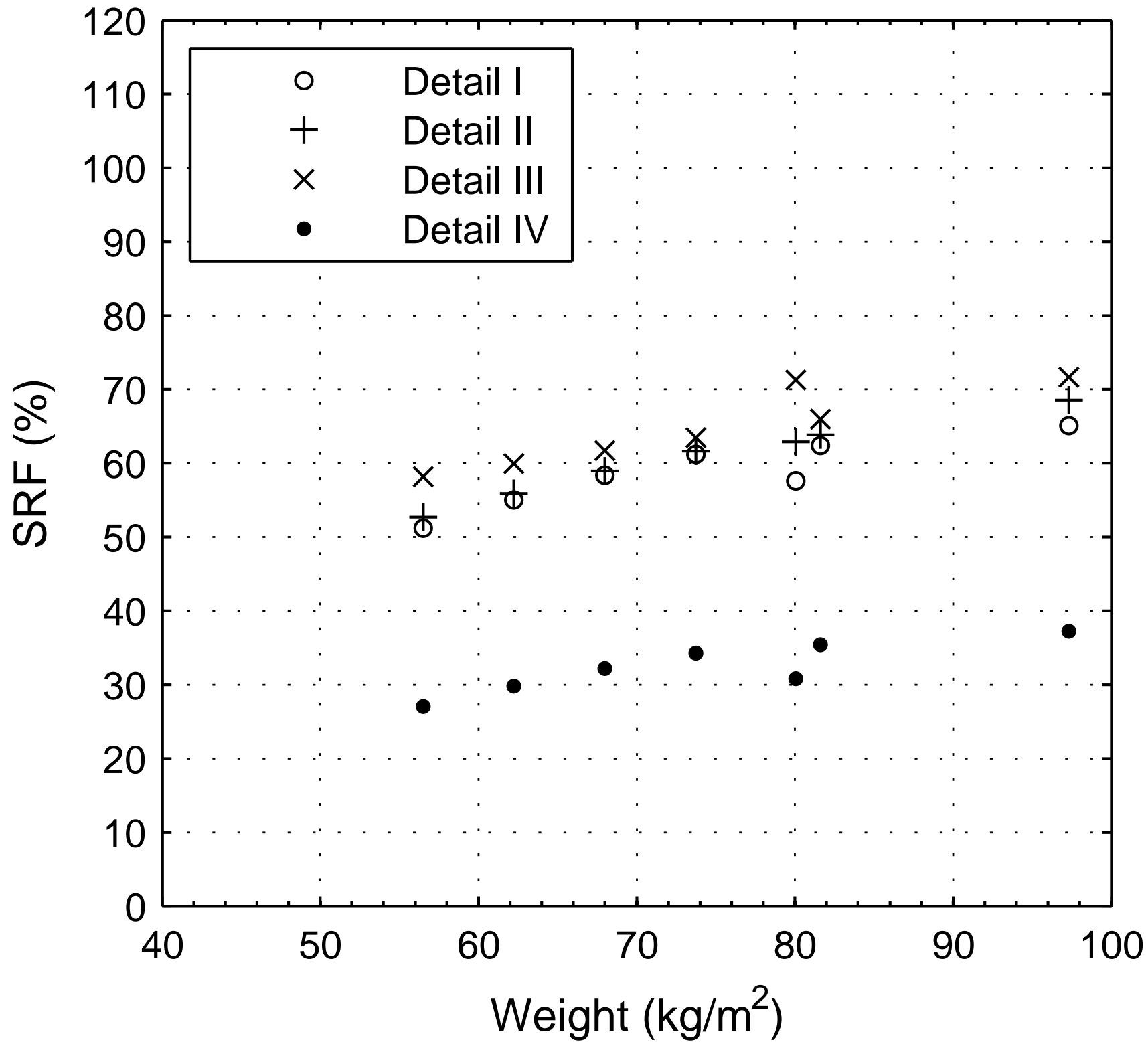


Figure 1- Geometry of the deck specimens (dimensions in mm). (a) Longitudinal cross section;
(b) Transverse cross section

Figure 2 – Retrofitting systems (dimensions in mm). a) Bonded system; b) Sandwich system

Figure 3 – Strain gauges location (dimensions in mm). (a) Midspan; (b) Crossbeam; (c) Top view
of the midspan; (d) Top view of the crossbeam

Figure 4 – Position of the strain gauges close to the deck-to-stiffener weld.

Figure 5- Example of a fatigue test performed at the crossbeam location (crossbeam A, trough 2).
(a) Longitudinal view; (b) Crossbeam A cross-section

Figure 6 – Experimental test set-up.

Figure 7 - Three-dimensional finite element model overview (Teixeira de Freitas 2013b).

Figure 8 - Transverse strains ε_{xx} at the crossbeam (CB) and 75 mm from the crossbeam (75 mm
CB) on the bottom side of the deck plate recorded during testing (Exp) and predicted by the FEA
(Teixeira de Freitas 2013b). a) bonded system, b) sandwich system

Figure 9 - Transverse strains ε_{xx} at midspan on the bottom side of the deck plate recorded during
testing (Exp) and predicted by the FEA (Teixeira de Freitas 2013b).

a) bonded system, b) sandwich system

Figure 10 – Strain ranges measured during fatigue tests of the bonded steel plates reinforced deck specimen.

(a) Example of strains at a crossbeam fatigue test: $P_{\max} = 160 \text{ kN}$ ($\Delta P = 144 \text{ kN}$).

(b) Strains at the midspan between crossbeams: $P_{\max} = 160 \text{ kN}$ ($\Delta P = 144 \text{ kN}$).

Figure 11 – Strain ranges measured during fatigue tests of the sandwich steel plates reinforced deck specimen

(a) Example of strains at a crossbeam fatigue test: $P_{\max} = 160 \text{ kN}$ ($\Delta P = 144 \text{ kN}$)

(b) Strains at the midspan between crossbeams fatigue test: $P_{\max} = 110 \text{ kN}$ ($\Delta P = 99 \text{ kN}$).

Figure 12 – Fatigue cracks in deck-plate-to-stiffener welds at the crossbeam location ($P_{\max}=160 \text{ kN}$, $\Delta P=144 \text{ kN}$). (a) Bonded steel plates reinforced deck. (b) Sandwich steel plates reinforced deck.

Figure 13 – Shear stress distribution τ_{xy} of the reinforced decks loaded at the middle trough by wheel type C at the crossbeam cross section or at midspan between crossbeams (100 kN).

(a) Shear stress in the adhesive layer (mid-thickness) of the bonded steel plates system.

(b) Shear stress at the steel-core interface (max between the two interfaces) of the sandwich steel plate system.

Figure 14– Comparison of the SN diagrams of the reinforcements in the full-scale fatigue tests

and in the bending fatigue tests. (a) bonded system, (b) sandwich system

Figure 15 – Comparison between the SN fatigue results of the welds at the crossbeam cross sections and the detail categories defined in EN 1993-1-9 (2005). (a) 10% strain fall criterion; (b) 25% strain fall criterion.

Figure 16 – Comparison between the SN fatigue results of the welds at midspan between crossbeams and the detail categories defined in EN 1993-1-9 (2005).

Figure 17 – Deck detail location (I to III transverse stresses and IV longitudinal stresses).

Figure 18 – Stress reduction factor (SRF) at the cross beam location for different reinforcement weight scenarios. (a) bonded system, (b) sandwich system.

Figure 19 – Stress reduction factor (SRF) at midspan between crossbeams for different reinforcement weight scenarios. (a) bonded system, (b) sandwich system.