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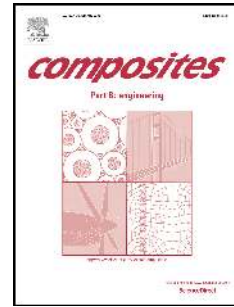
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Strengthening of RC beams using bottom and side NSM reinforcement

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Abstract

The allowable strain in fibre reinforced polymers reinforcement is limited by design codes to avoid debonding. The near-surface mounted (NSM) reinforcement technique has been proven to produce better anchorage behaviour compared to externally bonded reinforcement solutions. However, NSM solutions do not always eliminate debonding issues, with concrete cover detachment (CCD) typically occurring in RC beams strengthened for flexure. This experimental study investigated the efficiency of side mounted (S) compared to bottom mounted (B) NSM bars to prevent CCD. The experimental results were compared to models available in the literature that predict the observed failure modes and the crack spacing in the NSM anchorage zone. Compared to B-NSM, the S-NSM solution was successful in avoiding brittle CCD failure and showed increased rotational capacity and energy dissipation at failure. Existing CCD debonding models were found to be conservative.

Keywords: debonding; concrete cover detachment; crack spacing; non-contact optical measurements; strain analysis; CFRP; NSM

25 1. Introduction

26 Repair and strengthening with fibre-reinforced polymers (FRPs) is a well-established
27 rehabilitation method in the construction industry, with numerous design guidelines available
28 worldwide (e.g. ACI 440.2R [1], CSA S806 [2], CNR-DT200R1 [3], Fib Bulletin 14 [4], [5]) and
29 others soon to be introduced, such as [6].

30 Flexurally-designed reinforced concrete (RC) beams can fail because of yielding of the tension
31 reinforcement, concrete crushing, or shear flexure. Two of the most common flexural
32 strengthening methods are externally bonded reinforcement (EBR) and near-surface mounted
33 reinforcement (NSM). In both cases, the FRP reinforcement is bonded to the tension side of the
34 element. With the EBR technique, fabrics or laminates, are adhesively bonded directly to the
35 concrete surface whereas, with the NSM technique, FRP bars or lamella are inserted into grooves
36 cut in the element's concrete cover. When RC members are strengthened with FRP, additional
37 failure modes are possible, namely: (a) concrete cover detachment (CCD), (b) end interfacial
38 debonding, or (c) intermediate crack debonding (ICD) [7]. Sudden debonding failure modes were
39 observed in experimental investigations when EBR strengthening was used [8, 9]. This
40 phenomenon usually happens before reaching the tensile strength of the fibres, thus hindering the
41 utilisation of the FRP to the maximum capacity.

42 The NSM technique was, to the authors' knowledge, first applied in the mid 90-ties for
43 strengthening of a RC cable stayed bridge in Uddevalla, Sweden. Considerable research on NSM
44 strengthening solutions [8-19] has since been carried out. Compared to EBR, the NSM technique
45 provides increased interfacial stress capacity [8-10], due to a larger bond surface and confinement
46 provided by the surrounding concrete [9, 19]. Although the FRP utilisation is increased,
47 debonding failure modes are not avoided.

48 Current design codes provide special requirements regarding CCD. For example, ACI 440.2R
49 [1] specifies that transversal anchors should be provided at the FRP bar cut-off section if the shear
50 force in the section is more than 67% of the shear strength of the concrete section. Similarly,

51 CNR-DT200R1 [3] recommends the use of anchorage devices such as FRP U-wraps [20] or NSM
52 strips [21]. However, for use in practice, the performance of such solutions needs to be
53 determined experimentally.

54 In bottom mounted NSM (B-NSM) strengthened beams, CCD is likely to occur if the NSM is
55 terminated at a distance (Δl) from the support. For example, CCD was reported for relatively large
56 cut-off lengths ($\Delta l=200\text{mm}$) [11, 13] but also for small cut-off lengths ($\Delta l=50\text{ mm}$) [12].

57 To the authors' knowledge, only one study was carried out on RC beams strengthened with
58 side mounted NSM (S-NSM), reported in Hosen et al. [22] and Shukri et al. [23], and showed that
59 the S-NSM technique is effective for strengthening RC beams in flexure, in terms of serviceability
60 and ultimate load. The beams were strengthened with steel and carbon FRP (CFRP) rods of 8, 10,
61 and 12 mm diameter. The S-NSM technique produced a significant increase in flexural capacity
62 and cracking load with respect to the reference beams. However, beams strengthened with 12 mm
63 diameter steel and CFRP bars failed due to concrete cover detachment. Shukri et al. [23] further
64 investigated the influence of existing cracks on the performance of CFRP strengthened beams. It
65 was found that pre-cracking slightly decreased the beams' ultimate strength and increased beams'
66 rigidity, however, failure modes remained unchanged.

67 Often, access to the soffit of RC beams in need of strengthening is limited. Examples of such
68 cases are: a) beams part of a road crossing bridge; b) beams part of a building's structure directly
69 above industrial equipment; and c) beams or spandrels created by cutout openings in RC walls. In
70 such cases, when: a) the road is to remain open; b) the industrial equipment is to remain
71 operational; and c) strengthening of the beam is desired before cutting out the opening; the [more
72 usual] B-NSM technique becomes problematic to apply and the S-NSM technique could be used
73 in instead.

74 Despite the above-mentioned advantages of S-NSM over B-NSM, the efficiency of S-NSM
75 can be limited by the shorter lever arm in comparison to the B-NSM technique and the limited
76 amount of additional reinforcement that can be provided to the sides of the beams. However,

77 currently no direct comparison has been reported within the available literature. Moreover, the
78 bond behaviour of S-NSM strengthen RC beams is largely unexplored, and, as suggested in
79 Shukri et al. [23], investigations are required to evaluate the effect the bonded length on the
80 effectiveness of the S-NSM technique.

81 The experimental study, presented in this paper, investigated the efficiency of the S-NSM
82 technique compared to B-NSM, with varied bonded lengths, in terms of ultimate capacity, crack
83 distribution, stiffness, and failure modes. In this study, a 3D optical deformation measurement
84 system Aramis 5M [24], that utilises the digital image correlation (DIC) technique, was used to
85 measure deformations and identify cracks in the NSM anchorage area. The experimentally
86 obtained crack spacing was compared to predictions using existing analytical formulas, and was
87 used to evaluate the performance of existing debonding models.

88 2. Experimental programme

89 The experimental programme consisted of seven RC beams, each with a length of 4000 mm
90 and a 200 × 300 mm rectangular cross section (Fig. 1). One beam was tested as a reference
91 specimen; the other six beams were strengthened using different FRP configurations. The
92 longitudinal reinforcement consisted of two 16 mm diameter deformed steel bars, which were
93 placed at both the top and bottom part of the cross section, see Fig. 1. Shear reinforcement
94 consisted of steel stirrups made of deformed steel bars with a nominal diameter of 10 mm,
95 uniformly spaced at 75 mm.

96 The primary test variable was the placement of NSM (S-NSM vs B-NSM). In all cases, two
97 CFRP bars were used to strengthen the specimens. To produce a CCD, the CFRP bars ended at a
98 distance Δl from the beam's support.

99 B-NSM beams are denoted B300, B250 and B200 which corresponds to cut-off lengths (Δl)
100 300, 250 and 200 mm, respectively. S-NSM beams are denoted S300, S250 and S200 which
101 corresponds to values of Δl 300, 250 and 200 mm, respectively. Δl was varied only at one end,
102 north (see Fig. 1), to facilitate CCD failure only in one end, thus making it possible to monitor the

103 area expected to fail. At the other end of the beam, the CFRP bars were extended over the support
104 up to the beam's end.

105 The NSM groove size was 1.5 times the bar's size, as per ACI 440.2R [1]. For B-NSM
106 strengthened beams, the distance between grooves (i.e. 80 mm) was larger than twice the depth of
107 the groove (i.e. 30 mm). The clear edge distance was 15 mm for all strengthened beams, which
108 was smaller than recommended by ACI 440.2R [1]. However, this was chosen to minimise the
109 difference in effective depth between the B-NSM and S-NSM strengthened beams.

110 2.1 Material properties

111 The concrete compressive strength was determined from six cubes with standard sizes
112 according to the procedure described in [25]. The average cube compressive strength (f'_c) was
113 62.6 MPa equivalent to 50 MPa concrete cylinder compression strength (f_c), according to EC 2
114 [6]. The average yielding strength (f_y) and yielding strain (ϵ_y) of the longitudinal reinforcement
115 were 578 MPa, and 2.79‰, respectively, determined according to SS-EN ISO 6892-1 [26].

116 Rectangular (10 x 10 mm) CFRP bars (StoFRP Bar IM 10 C) and epoxy-based adhesive
117 (StoPox SK 41) were used. The CFRP bars had 3300 MPa nominal tensile strength and 210 GPa
118 modulus of elasticity. The adhesive had 12 MPa shear strength and 2GPa modulus of elasticity.
119 CFRP and adhesive material properties were taken according to the manufacturer's specifications.

120 2.2 Test set-up and instrumentation

121 The beams were loaded in a four-point bending set-up (see Fig. 1) with a span L_0 of 3600 mm,
122 using displacement control at a loading rate of 0.6 mm/min up to failure. Load (P), midspan
123 deflection (δ), steel strain (ϵ_s), and CFRP strain (ϵ_f) were measured during the loading. Two
124 displacement transducers were used to monitor δ , one at each lateral side of the beam. The strains
125 in the flexural steel and CFRP reinforcement were recorded with strain gauges placed at the
126 midspan, one on each reinforcement bar.

127 DIC was used to measure full field deformations and identify cracks in the anchorage zone on
128 the north side of the beam. DIC is a technique that uses digital camera images to measure shape

129 and displacement, and requires a contrast pattern to be able to determine the displacement of
130 subsets of the analysed images from the initial undeformed stage to the subsequent deformed
131 stages [27]. The images were acquired with two cameras having 2448 x 2050 pixel resolution and
132 equipped with 12 mm focal length lenses.

133 The positioning of the two cameras relative to the measured surface and to each other is shown
134 in Fig. 2. The cameras were mounted on a rigid crossbar at a 25° angle and spaced 600 mm apart.
135 The crossbar was placed on a tripod positioned one metre from the measured surface. For the S-
136 NSM beams, the cameras were placed next to the beam, perpendicular to the monitored area while
137 for B-NSM beams, the cameras were placed below the level of the soffit of the tested beams.
138 They were not placed directly under the beam, to avoid damage when the beam failed. This
139 resulted in a 60° angle between the direction of the cameras and the measured surface. Images
140 were acquired at a rate of one a second, and the applied load was recorded for each set of images.

141 3. Test results

142 The overall behaviour of the tested specimens was measured in terms of load-deflection
143 response, failure mode, steel and CFRP reinforcement response, and bending stiffness. In
144 addition, CCD of the NSM was investigated through a strain analysis by means of DIC
145 measurements. Table 1 shows a summary of the test results for all specimens.

146 To compare the performance of B-NSM and S-NSM strengthening, the following load levels
147 were identified from experimental tests and are shown in Table 1: (1) first crack P_{cr} , (2) steel
148 yielding load, P_y , and (3) ultimate load, P_u , for which the corresponding deflection, δ_u and
149 maximum strain in the CFRP bars, ϵ_{fu} , are indicated. It should be noted that P_{cr} was determined
150 based on slope changes in the P - δ responses and P_y was determined based on strain gauge
151 measurements. Table 1 also shows the energy dissipation at failure, E_d , computed as the area
152 under the P - δ graph for each beam, and the bending stiffness of the beam before the yielding of
153 the steel reinforcement K , computed as the slope of the P - δ curve between the cracking (P_{cr}) and
154 yielding (P_y) loads, respectively.

155 3.1 Reference beam

156 A typical trilinear response was observed for the reference beam from which the concrete
157 cracking, and steel yielding points could be identified (see Fig. 3). The beam failed by yielding of
158 the bottom steel reinforcement followed by concrete crushing of the top of the cross section.

159 3.2 Beams strengthened with B-NSM

160 Compared to the reference beam, the B-NSM strengthened beams exhibited a significant
161 increase in the yielding load (117 – 128% increase) and ultimate load (122 – 136% increase), see
162 Table 1. Fig. 3 shows the P - δ response of B-NSM strengthened beams compared to the reference
163 beam. Up to P_{cr} , the stiffness of B-NSM strengthened beams was identical to that of the reference.
164 Between P_{cr} and P_y , the stiffness of the strengthened beams was 65 – 76% higher. After P_y , the P -
165 δ became nonlinear, with the bending stiffness slowly decreasing up to failure (Fig. 3).

166 Thinner flexural cracks were observed compared to the reference beam. However, due to stress
167 concentrations, inclined cracks appeared from the NSM cut-off point. The cracks continued to
168 open until the concrete cover was detached together with the CFRP bars (see Fig. 4). Failure
169 occurred by CCD at the level of the steel reinforcement starting from the CFRP bars' cut-off
170 point, on the northern side, see Fig. 4. The observation was identical for all B-NSM beams,
171 regardless of their cut-off length. A slight increase in the maximum load was observed with a
172 decrease of Δl (see Table 1). The energy dissipation at failure was 41 – 73% higher than that of
173 the reference beam. The maximum strain in the CFRP bars was between 45% and 47% of ε_{fu} .

174 3.3 Beams strengthened with S-NSM

175 Compared to the reference beam, S-NSM strengthened beams exhibited a significant increase
176 in the yielding load (83 – 98% increase) and ultimate load (122 – 127% increase), see Table 1.
177 Fig. 5 shows the load-deflection (P - δ) response of S-NSM beams compared to the reference
178 beam. The cracking load, P_{cr} , of specimen S300 was not recorded. Due to a malfunction in the
179 data acquisition system, the test was stopped at a load level of 85 kN. The beam was then
180 unloaded and reloaded, thus the initial part of the response differs to the other strengthened

181 beams. Up to P_{cr} , the stiffness of S-NSM strengthened beams was identical to that of the
 182 reference. Between P_{cr} and P_y , the stiffness of the strengthened beams was 46 – 76% higher. After
 183 P_y , the bending stiffness decreased up to failure. Failure occurred due to concrete crushing at the
 184 top side of the beam, close to the south load application point (as shown in Fig. 6). ICD of the
 185 CFRP bars occurred only after the maximum load was reached, while the compressed concrete
 186 was still crushing. This debonding started at a flexural-shear crack with approximately 45°
 187 inclination. The debonding process ended when the CFRP slipped within the concrete groove (see
 188 Fig. 6). This behaviour was identical for all S-NSM strengthened beams. The maximum strain in
 189 the CFRP bars was between 51% and 54% of their ε_{fu} , as shown in Table 1.

190 4. Performance comparison between B-NSM and S-NSM

191 4.1 Loads and failure mode

192 Fig. 7 shows the comparison of the load-deflection response of B-NSM and S-NSM
 193 strengthened beams having the same Δl . In general, compared to S-NSM beams, B-NSM beams
 194 showed higher cracking loads (13%), bending stiffness (14%) and yielding loads (23%). These
 195 differences are due to the location of the CFRP bars, specifically the larger effective depth for B-
 196 NSM beams compared to S-NSM beams.

197 B-NSM strengthened beams exhibited a brittle CCD at the north anchorage side, whereas the
 198 S-NSM strengthened beams showed a more ductile failure by concrete crushing followed by ICD
 199 at the south end of the beam. The reason S-NSM beams did not fail by CCD at the north
 200 anchorage side can be attributed to the location of the CFRP bars relative to the flexural steel
 201 reinforcement (see Fig. 8). The concrete between the steel reinforcement and the NSM is
 202 subjected to tensile and shear stresses induced by the force transferred from the NSM to the end
 203 anchorage zone (F_f) [28]. Tensile stresses result from the moment $M_c = F_f \cdot l_c$, see Fig. 8. The
 204 distance l_c is defined as the distance between the NSM and the possible failure plane. For B-NSM,
 205 the failure plane is located at the lower level of the flexural steel reinforcement, whereas for S-
 206 NSM, a possible failure plane is located above the level of the internal reinforcement (see Fig. 8);

207 this aspect is further discussed in Section 4.2.3 End anchorage crack pattern. For B-NSM, l_c
208 promotes CCD, whereas for S-NSM, l_c leads to considerably lower tensile stresses. This would
209 suggest that CCD is not likely to occur in S-NSM strengthened beams. However, the experiments
210 reported in [22, 23] prove the opposite.

211 While similar ultimate loads were obtained for B-NSM and S-NSM strengthened beams with
212 the same Δl , the deflection at failure was 31 – 46% higher for S-NSM strengthened beams.
213 Comparing energy dissipation at failure, beams S300, S250 and S200 had 125%, 77%, and 62%
214 higher E_d than beams B300, B250, B200, respectively. Thus, B-NSM strengthened beams have a
215 higher bending stiffness overall while S-NSM strengthened beams have a higher ductility and
216 rotational capacity.

217 4.2 Strain analysis

218 4.2.1 Load-strain response

219 Compared to B-NSM beams, S-NSM beams' steel and CFRP reinforcement strains (Fig. 9s
220 and Fig. 9b, respectively) were initially slightly higher and lower, respectively. This is expected
221 due the difference between the effective depth of the CFRP reinforcement for the two systems.
222 After P_y however, the CFRP strain for S-NSM beams was higher. For beam S200, the steel strain
223 gauges malfunctioned at approximately 125 kN applied load.

224 The maximum CFRP strain in the S-NSM beams was 11 – 18% higher compared to B-NSM
225 beams. Thus, the CFRP reinforcement had a higher utilisation ratio (0.50 to 0.54 ε_{fu}) in the S-
226 NSM beam configuration compared to the B-NSM beam configuration (0.43 to 0.46 ε_{fu}).

227 4.2.2 End anchorage longitudinal crack spacing

228 The distribution of major principal strains in the monitored areas preceding failure is shown in
229 Fig. 10. For B-NSM beams, only the soffit of the beam was monitored while for S-NSM beams,
230 only the side of the beam was monitored. According to the coordinate system shown in Fig. 2, the
231 strain maps in Fig. 10 are given relative to the planes xy and xz for B-NSM and S-NSM beams,
232 respectively. In all cases, x represents the longitudinal axis of the beam with zero being the

233 location of the north support. Axis y represents the thickness of the beams, with zero being the
234 side of the beam. Axis z represents the height of the beam, with zero being the soffit of the beam.
235 It should be noted that the strains shown in Fig. 10 greatly exceed the realistic tensile strains
236 expected in the concrete. However, the presented strain distribution is useful for evaluating crack
237 initiations, paths, distribution and spacing. The scale representation of deformation in this case
238 was chosen such that red areas indicate fully formed cracks. For both B-NSM and S-NSM
239 strengthened beams, crack spacing was measured at the bottom face of the beam.

240 The minimum distance between two consecutive cracks (i.e. minimum crack spacing) observed
241 for B-NSM beams was approximately 50 – 60 mm. The maximum distances between two
242 consecutive cracks (i.e. the maximum crack spacing) of 80, 100, and 110 mm were observed for
243 B300, B250 and B200, respectively, closest to the cut-off point. The maximum crack spacing in
244 this case was observed to be approximately twice the minimum spacing. Moreover, Fig. 10
245 indicates that the cut-off length l possibly influences the crack spacing, a parameter currently not
246 accounted for by existing equations for predicting crack spacing in B-NSM beams [29-31].

247 Larger crack spacing was observed for S-NSM than for B-NSM beams, being 185, 180 and
248 170 mm for S300, S250 and S200, respectively. This suggests that the location of the NSM
249 (bottom or side) influences the stress distribution in the anchorage zone and consequently the
250 crack patterns.

251 4.2.3 End anchorage crack pattern

252 In S-NSM strengthened beams, horizontal cracks were observed above the NSM (see Fig. 10),
253 also above the flexural steel reinforcement (see Fig. 8). Cracks start as flexural-shear cracks and
254 propagate towards the cut-off point, indicative of CCD. This differs from previous reported
255 observations for S-NSM strengthened beams [22] and B-NSM strengthened beams [11-13], where
256 the CCD was initiated as a vertical crack at the CFRP cut-off point, propagating below the level of
257 the flexural steel reinforcement towards the middle of the beam.

258 In S-NSM beams, the possible debonding plane was located above the flexural steel
259 reinforcement (see Fig. 8) and, assuming it propagates as the same level over the thickness of the
260 beam (as in the case of B-NSM [11-13]), the shear reinforcement intersected by the failure plane
261 prevented CCD. Therefore, in the tested S-NSM strengthened beams, the failure plane was not
262 fully established.

263 5. Comparison of experimental and analytical results

264 The simplified analytical model proposed by An et al. [32] for rectangular beams was used to
265 calculate sectional stresses and deformations. The model is based on the following assumptions:
266 (1) linear strain distribution through the full depth of the beam; (2) small deformations; (3) the
267 concrete does not carry tensile stresses after cracking; (4) shear deformations are not considered
268 and (5) there is a perfect bond between the internal steel reinforcement and concrete, and NSM
269 and the concrete. The stress-strain relationship for CFRP reinforcement is linear elastic. The
270 stress-strain relationship for internal steel reinforcement is assumed elastic-perfectly plastic.
271 Hognestad's [33] parabola of an idealised stress-strain curve was used for concrete in
272 compression.

273 The strain and stresses in the FRP bars and steel reinforcement, as well as curvature at
274 midspan, were calculated using an incremental deformation technique, in which the strain in the
275 extreme concrete fibre is increased from 0 to $3000\mu\epsilon$, considered the ultimate useful concrete
276 strain. The strain in the steel and FRP reinforcement was calculated for each increment from a
277 cross sectional analysis to satisfy the force equilibrium and deformation compatibility conditions.
278 The moment and curvature were computed using the moment-curvature relationship, starting from
279 the strain in the extreme concrete fibre. Finally, the load-deflection response was derived. An
280 automated calculation procedure was developed to carry out the calculations. A good agreement
281 can be seen in terms of applied load-midspan displacement between the model and the
282 experimental tests (Fig. 11).

283 The bond between internal and NSM reinforcement and concrete was not modelled explicitly.
 284 Instead, for CCD, the models proposed by Al-Mahmoud et al. [28] and Teng et al. [34] were used
 285 to predict the failure load of strengthened specimens. According to a recent assessment of current
 286 guidelines [1-3], current formulations for NSM interfacial debonding provide conservative results
 287 with limited accuracy [35]. Instead, in this study, the model proposed by Mohamed Ali et al. [36]
 288 was used to predict the interfacial debonding failure load.

289 5.1 Interfacial debonding model

290 Mohamed Ali et al. [36] proposed two models for the debonding of NSM, using linear and
 291 bilinear interfacial bond characteristics, respectively. Both models were shown to have good
 292 accuracy compared to experimental data obtained from pull-out tests, especially for predicting the
 293 debonding load [36]. The simplified linear model is a closed-form solution, applicable to ICD.
 294 The maximum interfacial shear stress and slip are obtained from:

$$295 \quad \tau_f = 0.54\sqrt{f_c}h_f^{0.4}b_f^{0.3} \quad (1)$$

$$296 \quad \delta_f = 0.78\left(\frac{f_c^{0.27}}{b_g^{0.3}}\right) \quad (2)$$

297 where f_c is the concrete compressive strength, h_f and b_f are the height and width of the FRP
 298 reinforcement, respectively, and b_g is the groove size.

299 For cases where the anchorage length is longer than the effective bond length, the NSM
 300 debonding strength is:

$$301 \quad F_{deb}^{ICD} = \frac{\tau_f L_{per}}{\lambda} \quad (3)$$

302 where L_{per} is the total perimeter of the FRP-concrete interface at the end of L_e and λ is a
 303 constant:

$$304 \quad \lambda = \sqrt{\frac{\tau_f L_{per}}{\delta_f E_f A_f}} \quad (4)$$

305 where E_f and A_f are the Young modulus and the total cross sectional area of the FRP
 306 reinforcement, respectively.

307 The FRP strain associated to F_{deb}^{ICD} can be determined as:

$$308 \quad \varepsilon_f^{ICD} = \frac{F_{deb}^{ICD}}{A_f E_f} \quad (5)$$

309 The moment in the cracked section $M(x_{cr})$ where debonding is initiated can be determined
310 from the cross section analysis starting from ε_f^{ICD} .

311 5.2 Concrete cover detachment model

312 Teng et al. [34] proposed a strength model for predicting the strain in the FRP at the critical
313 cracked section when CCD occurs:

$$314 \quad \varepsilon_f^{CCD} = 10^4 \cdot \beta_{cs} \beta_{AE} \beta_{bod} b_{clear} \sqrt{f_c} \quad (6)$$

$$315 \quad \beta_{cs} = \left(\frac{4.5}{w_c^{0.3}} - \frac{l_c}{w_c} \right) \left(\frac{w_c}{100} - 0.1 \right) \quad (7)$$

$$316 \quad \beta_{AE} = \frac{1}{(A_f E_f)^{0.9}} \quad (8)$$

$$317 \quad \beta_{bod} = \left(\frac{b_{clear}}{D_t} \right)^{0.1} \quad (9)$$

318 where D_t is the sum of the diameter of all tension steel reinforcement bars placed closest to the
319 NSM, and $b_{clear} = b - D_t$, the clear width of the beam. In Eq. 6, β_{cs} accounts for the combined
320 effect of l_c and w_c ; β_{AE} accounts for the influence of the FRP axial rigidity and β_{bod} accounts for
321 the effect of reinforcement size and number. Starting from ε_f^{CCD} , the moment in the critical
322 cracked section can be determined from cross section analysis. According to Teng et al. [34], the
323 model gives better predictions compared to the other available models suggested by [28, 37]. For
324 the sake of brevity, the models proposed by [28, 37] are not shown here.

325 5.3 Evaluation of crack spacing

326 The crack spacing in the anchorage zone is of great importance for predictions based on the
327 concrete tooth model. All available CCD debonding models [28, 34, 37] were developed using
328 crack spacing values obtained from experimental observations [28] or from finite element analysis
329 [34]. According to De Lorenzis and Nanni [31], the minimum crack spacing in NSM strengthened
330 beams can be calculated by:

$$331 \quad s_c^{min} = \frac{A_e f_{ct}}{u_s \Sigma O_s + \tau_f \Sigma O_f} \quad (10)$$

332 where A_e is the area of concrete in tension, $u_s = 0.28\sqrt{f'_c}$ and is the local bond strength of the
 333 steel reinforcement, τ_f is the bond strength of the NSM, $\sum O_s$ is the total perimeter of the internal
 334 steel reinforcement, and $\sum O_f$ is the total perimeter of NSM. Zhang et al. [30] proposed a similar
 335 equation for s_c^{min} , however $u_f = 0.28\sqrt{f'_c}$ and the total NSM groove perimeter are used instead
 336 of τ_f and $\sum O_f$, respectively. Other equations for predicting the crack spacing of NSM
 337 strengthened beams have been proposed by Piyasena et al. [29]. For the sake of brevity, they are
 338 not shown here.

339 Table 2 shows a summary of the observed minimum crack spacing (w_c^{min}) in the anchorage
 340 zone using DIC (see Fig. 10) and the values obtained using the three mentioned models. It should
 341 be noted that all models were developed based only on B-NSM strengthened beams and neither of
 342 them explicitly accounted for the influence of d_f or Δl .

343 Models by Zhang et al. [30] and De Lorenzis and Nanni [31] yielded results that were in better
 344 agreement with the minimum crack spacing observed for B-NSM beams, albeit the latter slightly
 345 overestimated, while values provided from the model by Piyasena et al. [29] were comparable
 346 with the ones observed for S-NSM beams. Perhaps the crack spacing of S-NSM beams can be
 347 evaluated using the same equation as that for internally reinforced beams, adapted to account for
 348 the mechanical and bond properties of NSM.

349 5.4 Comparison with test results

350 For four-point bending, the moment at the critical crack location (x_{cr}) can be obtained as:

$$351 \quad M(x_{cr}) = M \frac{x_{cr}}{a_v} \quad (11)$$

352 where M is the moment in the maximum moment region (i.e. between the load application
 353 point) and a_v is the shear span of the beam. From static equilibrium conditions, it follows that the
 354 associated total force in the actuator is:

$$355 \quad P = 2 \frac{M}{\alpha_L L_0} = 2M(x_{cr}) \frac{1}{x_{cr}} \quad (12)$$

356 where, for CCD debonding, x_{cr} can be estimated as $\Delta l + w_c$ and for ICD can be considered as
357 being $a_v - d_f/2$.

358 Table 3 shows a summary of the experimentally and analytically determined maximum loads
359 associated with CCD and ICD debonding failure. The predicted ICD debonding loads for B-NSM
360 beams were higher than the observed maximum load, which is in good agreement with the
361 observed failure modes, as B-NSM beams failed by CCD. The predicted CCD debonding loads
362 for B-NSM (calculated using [34]) and ICD debonding loads for S-NSM (calculated using [36])
363 are shown in Fig. 12, respectively, together with the experimental and theoretical load-deflection
364 responses.

365 Based on the experimental results for B-NSM beams, the models by Teng et al. [34] and
366 Hassan and Rizkalla [37] show similar performances, having average predicted-to-tested ratios of
367 0.67 and 0.66, respectively, whereas the model by Al-Mahmoud et al. [28] was the most
368 conservative, having an average predicted-to-tested ratio of 0.52. The model by [37] was found to
369 have the lowest coefficient of variation (COV), 0.03, whereas those by Teng et al. [34] and Al-
370 Mahmoud et al. [28] had 0.20 and 0.3 COV, respectively. In all cases, the experimentally
371 determined crack spacing for each beam was used as an input parameter. This indicates that the
372 former two models have a high sensitivity to the parameter Δl in the range investigated in this
373 study.

374 The ICD model proposed by Mohamed Ali et al. [36] predicted the capacity of S-NSM beams
375 with good accuracy, having an average of predicted-to-tested ratio of 1.01 and with 0.01 COV
376 (see Fig. 12b and Table 3).

377 For S-NSM beams, the models by Teng et al. [34] and Hassan and Rizkalla [37] provided
378 average predicted-to-tested ratios of 1.01 and 0.55, with 0.19 and 0.07 COV, respectively. While
379 CCD debonding did not occur, results suggest that models by both Teng et al. [34] and Hassan
380 and Rizkalla [37] provided conservative predictions. However, experimental tests where the CCD
381 failure mode for S-NSM beams occurs are necessary to determine the influence of shear

382 reinforcement, crack spacing in the anchorage zone, and NSM cut-off distance on the debonding
383 capacity.

384 **6. Conclusions**

385 This paper describes the results of experimental tests carried out on six RC concrete beams
386 strengthened with NSM CFRP reinforcement at different locations. The NSM were placed at the
387 bottom of the beam (B-NSM) and at the side of the beam (S-NSM). The NSM was provided with
388 different anchorage lengths. Strains and crack patterns in the anchorage zone were recorded using
389 a 3D-DIC deformation measurement system. These observations provided insight into the
390 concrete cover detachment failure mode. The performances of analytical models used to predict
391 the crack spacing in the anchorage zone and the maximum capacity of the strengthened beams
392 were investigated. Based on the findings of this study, the following conclusions can be drawn.
393 An S-NSM strengthening system, when compared to a B-NSM system:

- 394 ▪ Provided a similar ultimate load carrying capacity
- 395 ▪ Was successful in avoiding concrete cover detachment
- 396 ▪ Increased the CFRP strain at failure by 11 to 18%
- 397 ▪ Increased the energy dissipation at failure by 1.6 to 2.3 times
- 398 ▪ Increased the maximum deflection at failure by 31 to 46%

399 Moreover, by comparing experimental tests with models, the following conclusions can be drawn:

- 400 ▪ The interfacial debonding model by Mohamed Ali et al. [36] predicted, with good accuracy,
401 the intermediate crack debonding failure load of S-NSM strengthened beams
- 402 ▪ The crack spacing in the anchorage zone of B-NSM and S-NSM strengthened beams was
403 best predicted with models by Zhang et al. [30] and Piyasena et al. [29], respectively
- 404 ▪ Existing CCD debonding models for B-NSM strengthen beams were found to be
405 conservative, and future CCD debonding for S-NSM strengthened beams should consider
406 the influence of the beam's internal shear reinforcement

407 Further experimental tests are needed to study concrete cover detachment failure modes in S-
408 NSM strengthened beams. The influence of parameters such as concrete cover thickness, flexural
409 and shear reinforcement ratio, NSM location, and support conditions should be addressed in
410 future studies.

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496
497

498 **Figure captions**

499 Fig. 1. Test specimen details (units in mm)

500 Fig. 2. Test setup and location of DIC monitored area with respect to the defined coordinate
501 system

502 Fig. 3. Load-midspan deflection response of B-NSM beams

503 Fig. 4. CCD debonding of B-NSM strengthened beams

504 Fig. 5. Load-midspan deflection response of S-NSM beams

505 Fig. 6. S-NSM strengthened beams after failure (concrete crushing and ICD)

506 Fig. 7 Comparison of load-midspan displacement responses: (a) $\Delta l=300$ mm; (b) $\Delta l=250$ mm; (c)
507 $\Delta l= 200$ mm508 Fig. 8 Isolated concrete tooth between the last two adjacent cracks for B-NSM and S-NSM
509 strengthening system

510 Fig. 9 Load-strain response for (a) steel reinforcement: (b) CFRP bars

511 Fig. 10 Principal tensile strain distribution on the surface of the north anchorage zone, bottom
512 view for B-NSM (left) and side view for S-NSM (right), relative to the coordinate system
513 (units in mm)514 Fig. 11 Comparison between the theoretical and experimentally obtained Load-midspan deflection
515 response516 Fig. 12 Experimental and analytical comparison of debonding loads for a) B-NSM (CCD [34])
517 and b) S-NSM (ICD [36])

518

519 **Tables**520 *Table 1 Test results*

Specimen	Cracking load	Yielding load	Ultimate load	Max. deflection	Bending stiffness	Energy dissipation	Max. CFRP strain
	P_{cr} (kN)	P_y (kN)	P_u (kN)	δ_u (mm)	K (N/mm)	E_d (kNm)	ε_f ($\mu\text{m/m}$)
Ref	10.0	54	78	51.0	2643	2.79	-
B300	12.4 (24%)	118 (119%)	173 (122%)	40.6	4644 (76%)	3.94 (41%)	6390 (0.46 ε_{fu})
B250	16.0 (60%)	117 (117%)	178 (128%)	45.7	4355 (65%)	4.69 (68%)	6118 (0.44 ε_{fu})
B200	15.7 (57%)	123 (128%)	184 (136%)	45.7	4555 (72%)	4.82 (72%)	6526 (0.47 ε_{fu})
S300	n.a.	107 (98%)	177 (127%)	59.5	n.a.	n.a.	7114 (0.51 ε_{fu})
S250	14.6 (46%)	99 (83%)	177 (127%)	60.0	3978 (51%)	8.28 (198%)	7244 (0.52 ε_{fu})
S200	13.6 (36%)	102 (89%)	173 (122%)	60.5	3860 (46%)	7.83 (181%)	7568 (0.54 ε_{fu})

n.a. - data not available;

Note: increase relative to reference beam given in parenthesis.

521

522 *Table 2 Minimum crack spacing in the CFRP anchorage zone*

Specimen	s_c^{min}			w_c^{min}
	Piyasena et al. [29]	Zhang et al. [30]	De Lorenzis and Nanni [31]	
	(mm)			(mm)
B300				50
B250				50
B200	137	54	78	60
S300				^{b)} -
S250				120
S200				100

^{a)}Values from tests^{b)}The stabilised crack region was outside the DIC measured area

523

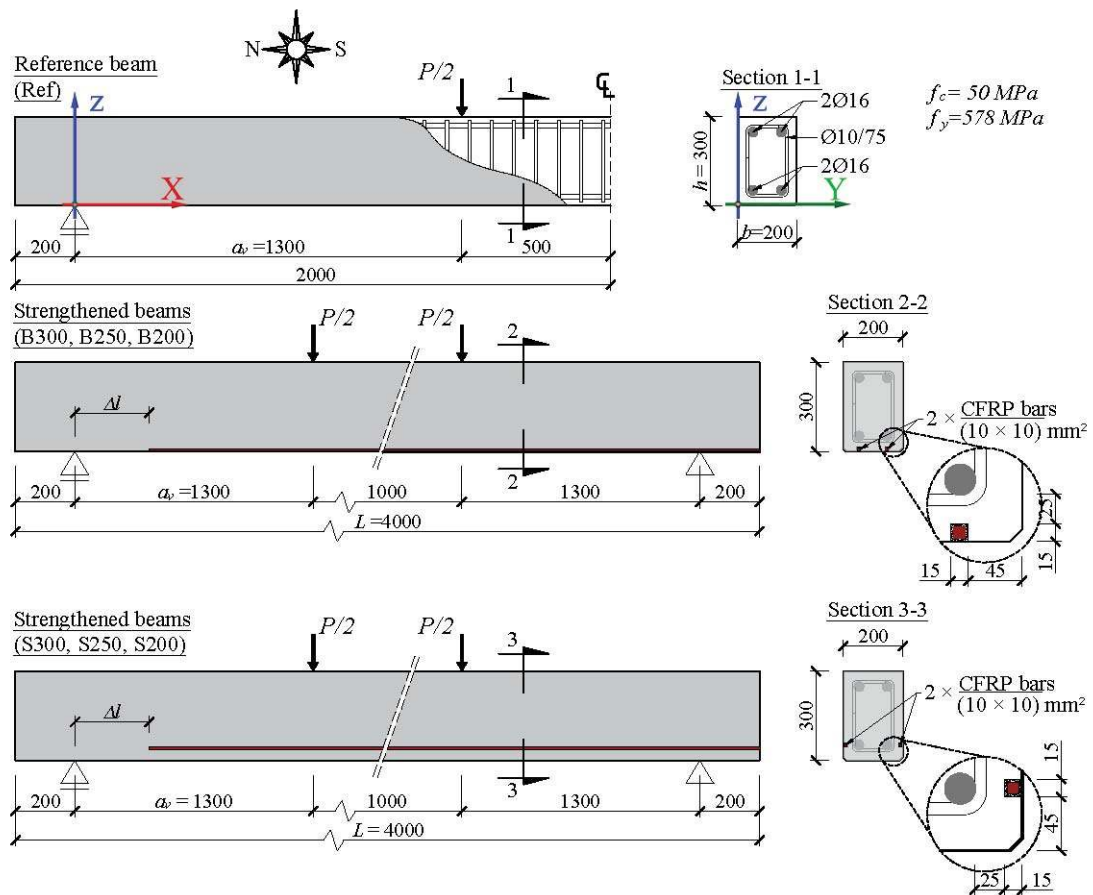
524 *Table 3 Experimental and analytical loads of strengthened beams*

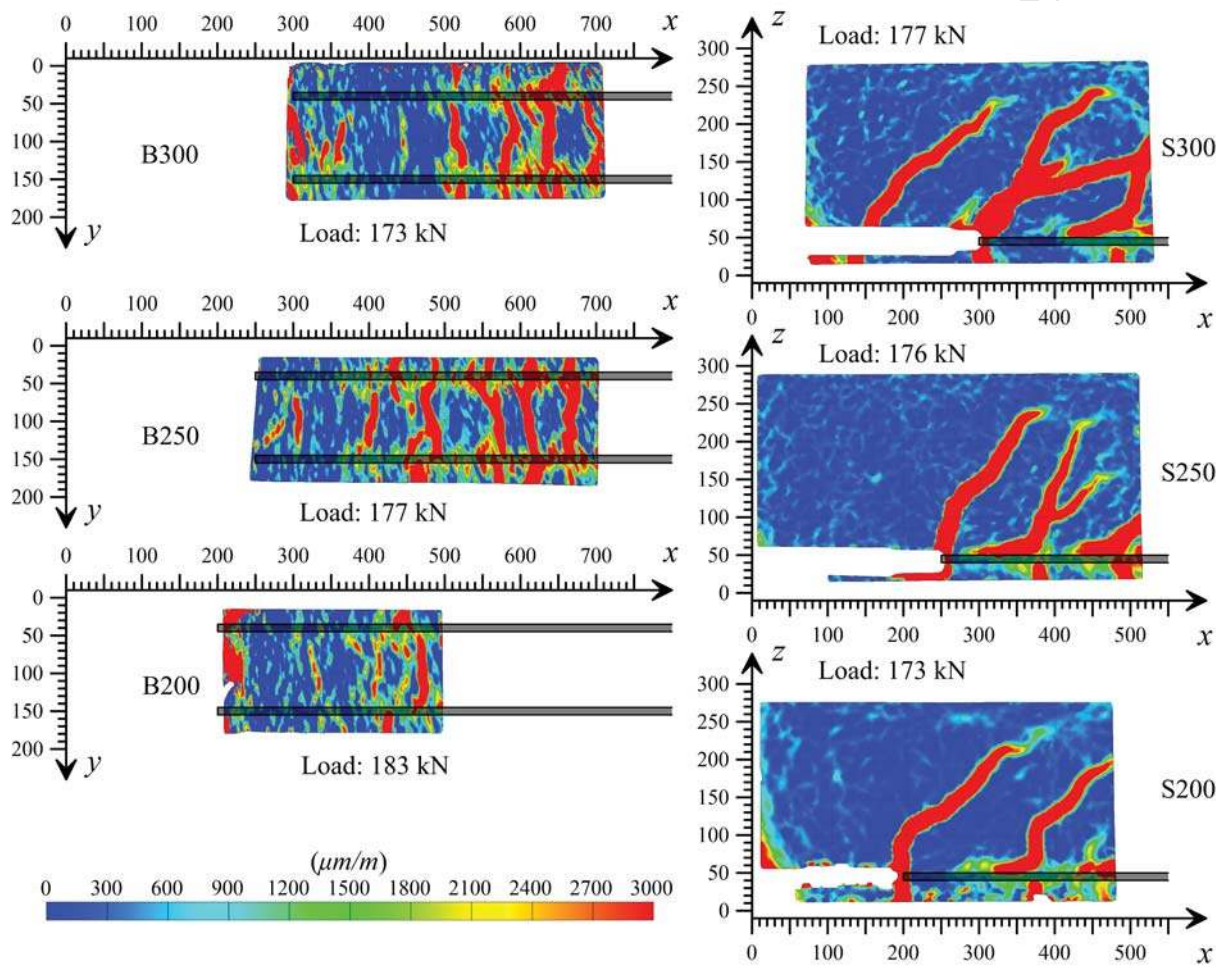
Specimen	Failure mode	P_{max}^{exp} (kN)	P^{CCD}		P^{ICD}		P^{CCD}/P^{exp}		P^{ICD}/P_{max}^{exp}	
			[34]	[37]	[28]	[36]	[34]	[37]	[28]	[36]
		(kN)	(kN)	(kN)	(kN)	(kN)				
B300	CCD	173	86	110	56	189	0.50	0.64	0.32	0.92
B250	CCD	178	116	120	95	189	0.65	0.67	0.53	0.94
B200	CCD	184	157	125	129		0.85	0.68	0.70	0.97
S300	ICD	177	135	90			0.76	0.51		1.02
S250	ICD	177	186	96		174	1.05	0.54		1.02
S200	ICD	173	210	104			1.21	0.60		1.00

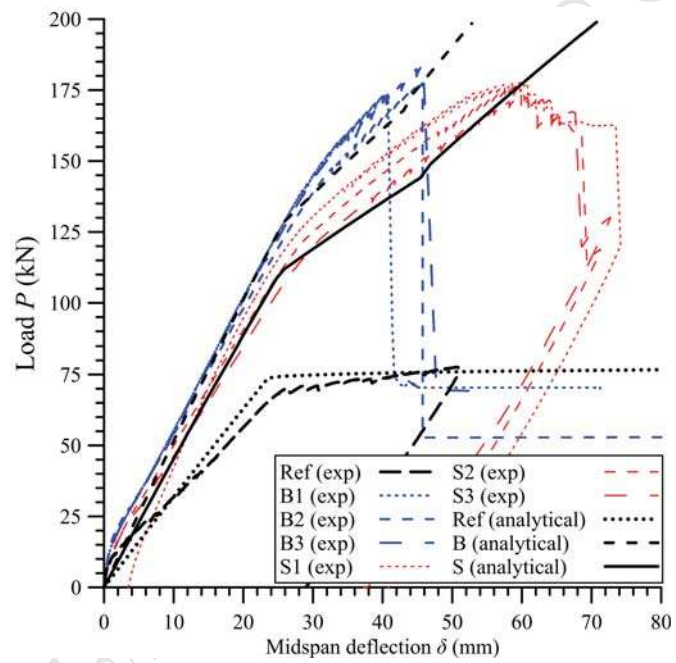
CCD Models: [Teng et al. \[34\]](#); [Hassan and Rizkalla \[37\]](#); [Al-Mahmoud et al. \[28\]](#);ICD Model: [Mohamed Ali et al. \[36\]](#)

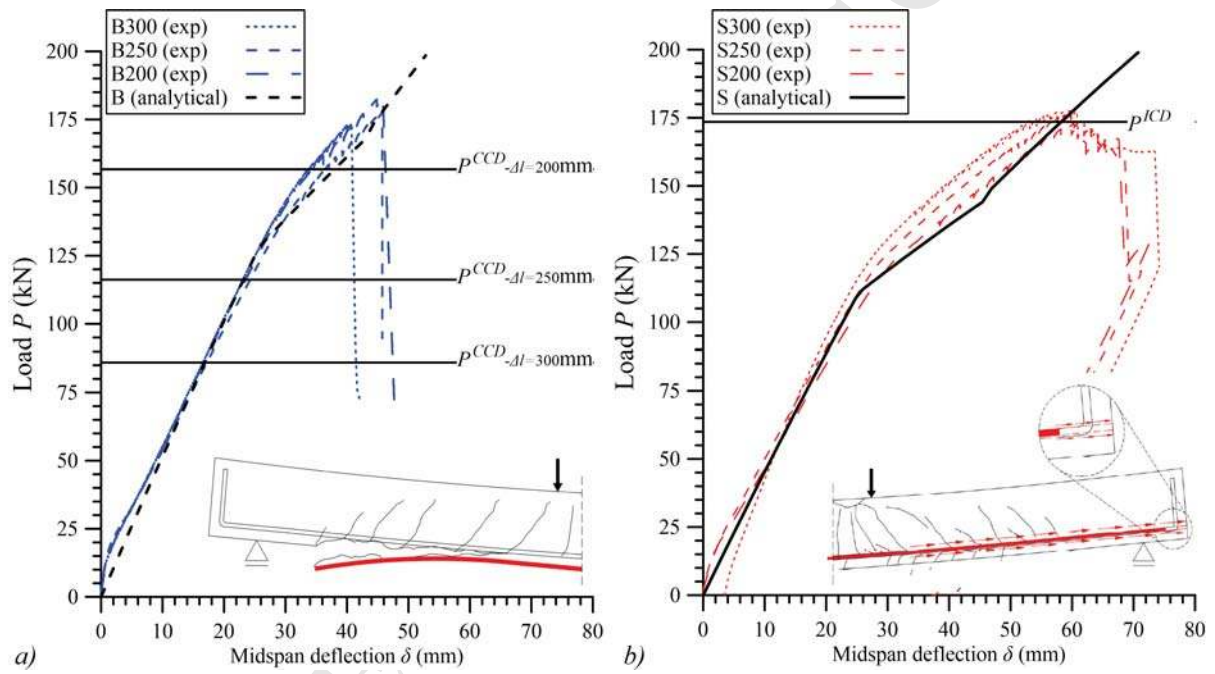
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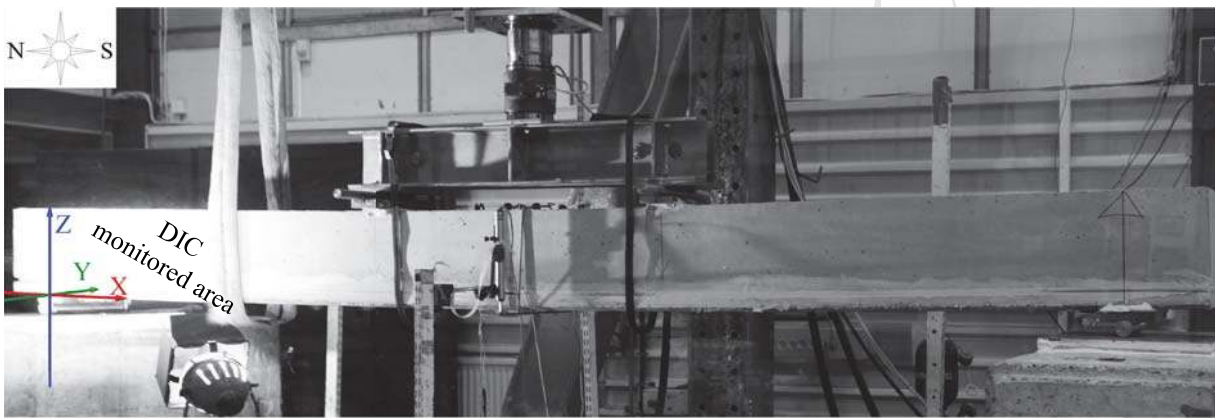
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B-NSM monitored area

S-NSM monitored area

