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Optimized seismic retrofit of steel-concrete composite buildings

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Abstract

This work is focused on comparatively assessing the cost-effectiveness of three seismic retrofit approaches for non-code-conforming frame buildings with steel-concrete composite columns. The first two of the assessed retrofit approaches aim in indirectly enhancing structural system performance by strengthening individual composite columns using reinforced concrete jackets or concrete-covered steel cages. The third retrofit approach considered aims in upgrading the composite building frame at hand by installing steel bracings at selected bays. A specially developed structural optimization procedure is used to perform an objective comparison of the cost-effectiveness of the three retrofit approaches. The objective of the optimization procedure is to minimize the total retrofit material cost, while constraints are imposed to ensure the satisfaction of design requirements for the retrofitted structure regarding member capacities (according to Eurocodes 3 and 4 for steel beams and composite columns, respectively), structural system performance under horizontal loading (based on interstorey drifts calculated by pushover analyses) and fundamental periods (obtained from eigenvalue analyses). By defining 30 cases of under-designed 2-storey, 4-storey and 6-storey composite buildings (*i.e.* buildings with steel-concrete composite columns), an extensive numerical investigation involving 120 retrofit optimization runs was conducted. The results obtained provide insight into the relative cost-effectiveness of the three seismic retrofit approaches and reveal certain conditions under which each approach is economically most viable.

Keywords: strengthening; concrete jacket; steel cage; steel bracing; cost-effectiveness; structural optimization.

1. Introduction

Steel-concrete composite design has often been utilized for structures intended for long lifetimes in areas with considerable or even high seismicity. However, such buildings constructed some decades ago generally do not conform to the provisions of current design codes (especially of seismic codes), which are significantly more demanding than the guidelines that were available at the time of their construction. In order to improve these buildings' seismic performance, they need to be appropriately retrofitted. There are two main approaches to retrofit a structural system: (a) strengthening of existing deficient members of the system and (b) installation of additional elements in the system. For both approaches, numerous methods have been proposed, investigated and applied in practice. Even though most of these methods are already considered to be well established, research is still ongoing, in an effort to improve their efficiency or numerical modelling, while also new methods involving innovative materials and techniques are presented.

The first retrofit approach involves the enhancement of the structural behaviour of individual members, joints or specific areas of the structure; therefore, this can be considered as a local approach. Strengthening methods following this approach aim at increasing the flexural capacity, shear capacity or ductility of particular structural elements and, consequently, enhancing indirectly the seismic resistance of the overall structural system. Various relevant techniques can be found in the literature, with the most well-known being reinforced concrete jacketing, steel jacketing and confinement with Fibre Reinforced Polymers (FRP).

Reinforced concrete jacketing is realized by installing additional reinforcement and concrete cover around an existing member. It is typically utilized to retrofit elements that are expected to have brittle failure and need an increase in their flexural capacity. It is a popular seismic retrofit method that has been actively researched and applied in the past (e.g. [1-3]) and attracts also more recent scientific interest, with researchers investigating it experimentally [4-9], analytically [10-12] or with the use of structural analysis software [10,13-15]. The focus of recent research efforts appears to be on the better modelling of the interface between the existing element and the concrete jacket, the improvement of analytical methods and the use of new materials.

Steel jacketing is another method used to strengthen individual structural members and increase their deformability and is realized by adding steel jackets around the retrofitted elements. This

2

method exploits the nearly full confinement of the existing concrete and the high flexural and shear capacity of the jacket's structural steel to achieve a more ductile behaviour for elements where failure is expected to be brittle [16,17]. Its aesthetic effect on the retrofitted building is minimal due to the reduced thickness of the steel jacket compared to reinforced concrete jackets. If the steel jacket is left exposed, it is more susceptible to fire hazard than the reinforced concrete jacket, in which the installed reinforcement is protected by the cover concrete. When the element's flexural capacity does not require to be enhanced, the steel wrapping can be placed only in specific critical areas where plastic hinges are expected to develop. As a variant of this retrofit method, individual steel plates are added to steel and steel-concrete composite beams and steel joints to enhance their capacity. Within the same context, steel plates and beams with I-shaped or U-shaped sections are used to retrofit also reinforced concrete beams [18,19]. Full steel jacketing can be analogously applied in practice in the form of interconnected steel strips and angles, forming a steel cage. The application of steel cages is roughly as old as that of reinforced concrete jacketing and is a field of active research efforts as well [3,20-25].

The second retrofit approach focuses on the overall improvement of a structure as a system, in order to meet the applicable seismic design requirements, therefore this can be characterized as a global approach. New elements are installed to limit the drifts and decrease the ductility demands of the structural system. Such seismic retrofit methods used in structural engineering practice include the integration of new shear walls, steel bracings, base isolators or dampers into an existing structure to achieve the desirable performance. A particularly popular method in practice is the installation of steel bracings in specific bays of the structure. The design of braced frames is nowadays common practice in buildings with steel and steel-concrete composite columns, but they have been used in reinforced concrete structures as well. Research on the utilization of steel bracings as a retrofit method started decades ago and is still active [26-36].

Although the aforementioned methods have been proposed, extensively evaluated and applied for the seismic retrofit of reinforced concrete and steel structures, their effectiveness in the case of steel-concrete composite structures has not yet been thoroughly investigated. Thus, the design and application of retrofit measures for composite structures based on these methods is strongly related to the particular engineer's judgment and experience from applications for other structural systems. In the present work, three methods are evaluated with respect to their cost-

96 effectiveness in retrofitting non-code-conforming multi-storey buildings with steel-concrete
97 composite columns. The decision on whether to retrofit a structure or demolish and replace it
98 is predominantly affected by the total cost of the intended retrofit solution. However, the
99 selection of a retrofit solution typically depends on the engineer's subjectivity. Hence, structural
100 optimization [37-41] is employed herein as an objective decision support tool that automatically
101 identifies the best solution on the basis of a 'fair' assessment, as it employs an optimization
102 algorithm to determine the most cost-effective feasible solution according to each retrofit
103 method evaluated. For this purpose, a structural optimization framework presented in [40] is
104 adjusted to the needs of the present work and applied to various cases of 2-, 4- and 6-storey
105 buildings with steel-concrete composite columns in need for seismic retrofit measures.

2. Retrofit methods

107 In this section, the retrofit methods evaluated in the present work are presented and details
108 regarding their implementation in the analysed composite building cases are given.

2.1. Retrofit of individual elements – Column strengthening

110 The local approach of retrofitting individual elements is followed in this work by two methods,
111 which are employed to strengthen steel-concrete composite columns, in order to indirectly
112 improve the buildings' seismic performance: reinforced concrete jacketing and concrete-
113 covered steel caging, herein referred to as 'jacket method' and 'cage method', respectively.
114 Both methods have been extensively used in practice to increase the ductility and flexural
115 capacity of deficient or damaged reinforced concrete columns. Their concepts are similar: both
116 are applied by forming a concrete-covered external grid of longitudinal and transversal steel
117 components, which confine the initial and additional concrete of the column, enhancing its
118 ductility and capacity. Additionally, if the longitudinal steel parts are adequately anchored to
119 develop their full capacity, the particular retrofit methods also increase the initial section's
120 stiffness. The main attributes that are different in the two methods are the type of steel elements
121 (bars/strips) comprising the grid that surrounds the initial member, as well as the
122 position/distance of these elements with respect to the centreline of the initial member.

123 Eurocode 4 [42] specifies that the contribution ratio δ of structural steel in the resistance of a
124 composite column section to compression should range between 0.2 and 0.9. This requirement
125 was checked and found to be satisfied for all fully encased steel-concrete composite column

126 sections considered in the present study. Clearly, by adding material(s) around the initial
127 sections, their composite behaviour can be significantly altered, shifting the ratio δ closer to
128 one of the two limit values mentioned above. In the jacket method, the section's extra concrete
129 area added for the retrofit is much larger than the additional steel area of the longitudinal
130 reinforcing bars within the jacket. In the cage method, more structural steel is added for the
131 retrofit to achieve the desirable stiffness, while concrete constitutes only the thin cover layer
132 and the patches between the horizontal and vertical strips; taking also into consideration that
133 the concrete cover is unconfined, the concrete's contribution to the extra capacity of the section
134 is rather low. Hence, with a thick reinforced concrete jacket, the composite section's behaviour
135 resembles more that of a reinforced concrete section (low δ-value), while a steel cage with wide
136 vertical plates increases significantly the contribution of structural steel to the composite
137 section's capacity (high δ-value).

2.1.1. Reinforced concrete jacketing (jacket method)

139 The installation of a reinforced concrete jacket around a column is one of the most commonly
140 used methods in practice to retrofit existing reinforced concrete columns, as well as one of the
141 most intensively investigated in literature. In this method, the extra concrete cover layer added
142 is unconfined, but the rest of the jacket's concrete forms an additional layer of confined concrete
143 core that enhances the column's capacity. Also, the jacket confines the initial element, creating
144 this way a core of super-confined concrete, which increases the column's capacity even further.

145 The jacket method is illustrated in Fig. 1, which shows the locations of the longitudinal bars
146 and the cage formed around the initial composite column. Since failure of the buildings
147 simulated in this investigation was found to be caused mainly by exceedance of the elements'
148 capacity in bending moment, the longitudinal bars of the jacket have a significant effect on the
149 retrofitted section's capacity. Hence, various diameters of bars are considered herein for the
150 longitudinal reinforcement. The effective stiffness added by the jacket is also directly related to
151 the developed capacity of the transversal reinforcement especially at the area of splices, where
152 careful design is needed. In this work, the transversal reinforcement applied is the same for all
153 simulated jacket sizes. The contribution of the added jacket section is analogous to that typically
154 obtained when retrofitting a reinforced concrete column: the longitudinal and transverse
155 reinforcements surrounding the concrete contribute to the section's bending moment and shear
156 capacity, respectively. The longitudinal reinforcement consists of 3 to 5 bars per column side,

157 depending on the dimensions of the initial element, while the transverse reinforcement consists

158 of rectangular stirrups that travel around the column (Fig. 1). The same effective concrete cover
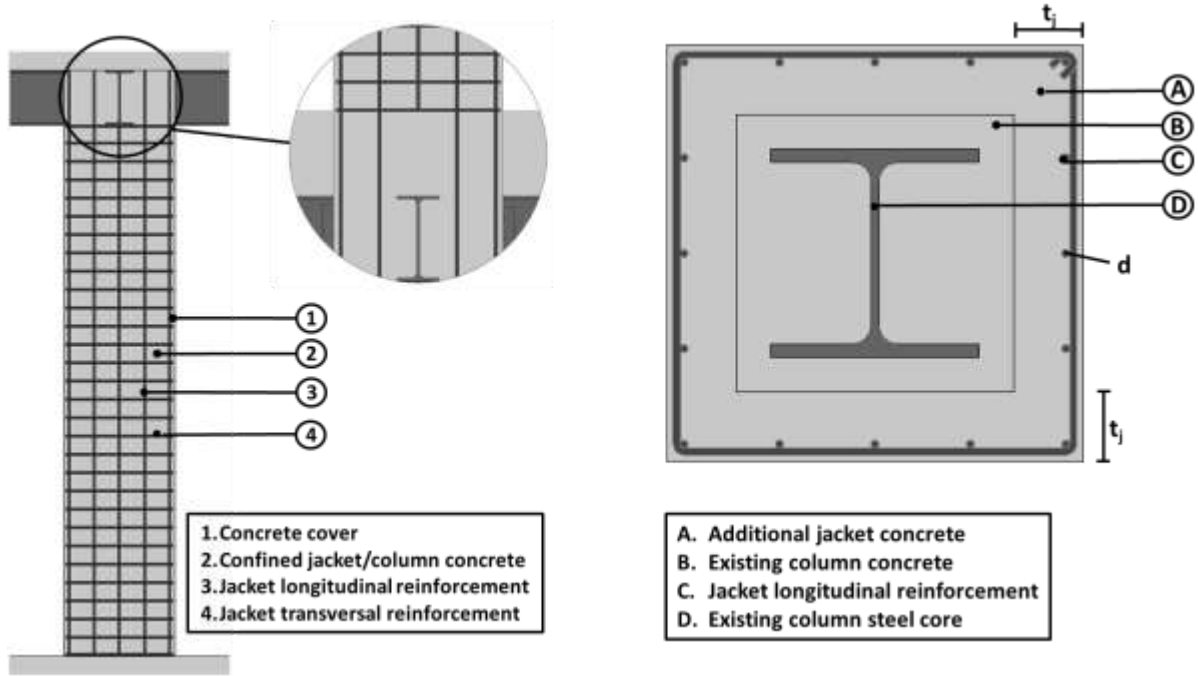
159 thickness of 2.5 cm is applied in all analysed cases.

1. Concrete cover
2. Confined jacket/column concrete
3. Jacket longitudinal reinforcement
4. Jacket transversal reinforcement

A. Additional jacket concrete
B. Existing column concrete
C. Jacket longitudinal reinforcement
D. Existing column steel core

160

161 **Fig. 1.** Column strengthening using reinforced concrete jacketing (jacket method): side view (left) and cross-
162 section (right) of the retrofitted column.

163 Since the thickness of the cover concrete, the number of longitudinal bars per column side and

164 the transverse reinforcement in this work are pre-specified for all analyses performed, only 2

165 parameters are required to define the reinforced concrete jacket applied to a composite column:

166 (a) the total thickness of the jacket per side (denoted as t_j in Fig. 1) and (b) the diameter d of the

167 longitudinal bars. In this work, a set of 18 pairs of jacket thicknesses and bar sizes defines the

168 available discrete options, from which a jacket can be selected for retrofitting a column or group

169 of columns. In particular, the first pair provides the option of not retrofitting (no jacket applied).

170 This option allows the optimization algorithm to selectively apply jackets only at specific

171 groups of columns, in order to attain the optimum retrofit design. The second pair is a theoretical

172 option resembling the application of the cage method: the total jacket thickness per column side

173 is set to 7.5 cm and the bar diameter to 50 mm, thus the jacket bars are placed in direct contact

174 with the existing column, while the additional concrete serves as cover and patches between the

175 bars. Use of this option in an optimized design would be indicative of a preference of the

176 optimizer for retrofit solutions following the cage method. The remaining pairs are

6

177 combinations of various standard bar diameters (up to 32 mm) for longitudinal reinforcement

178 with jacket thicknesses of 10 cm, 15 cm, 20 cm and 25 cm. It should be noted that large

179 reinforcement bar diameters (>20 mm) are options typically not selected for jackets in practice.

180 Nevertheless, they are made available to the optimizer, because their use in an optimum design

181 is an indicator of need for increase of the corresponding element's stiffness even further than it

182 is actually possible with conventional practices.

183 **2.1.2. Concrete-covered steel caging (cage method)**

184 Full steel jacketing is typically applied in deficient or damaged reinforced concrete columns at

185 locations where plastic hinges are expected to develop. The jacket confines the existing

186 concrete, increasing this way the column's ductility and shear capacity and improving its overall

187 performance. When the steel jacket is wide enough, full confinement of the existing concrete

188 can be achieved, producing an effect that is similar to the one observed in concrete-filled tubes,

189 in which the confined concrete can receive stresses significantly higher than its characteristic

190 capacity. However, the main drawback of the steel jacketing technique is the difficulty in

191 installation, as the steel section needs to be placed in at least two separate parts, which are then

192 welded together, in order to surround the existing element. Moreover, when composite action

193 of the jacket and the existing element is also required, additional dowels need to be installed to

194 operate as shear connectors.

195 The concrete-covered steel cage, which consists of vertical and horizontal plates, can be seen

196 as a hybrid retrofit method, which combines the concrete confinement effect achieved by the

197 installation of steel jackets and the increase of flexural capacity attained by welding longitudinal

198 steel plates on the flanges of I-shaped steel sections. Furtermore, its application in steel-concrete

199 composite columns aims to combine the effectiveness of full steel jaketing with the advantages

200 of the reinforced concrete jacket. In particular, the horizontal plates are mainly intended for

201 increasing the shear capacity and confinement of the concrete, adequately substituting the full

202 steel jacket, while the vertical plates aim mainly at increasing the column's stiffness and

203 flexural capacity. Because the steel plates are placed at larger distances from the composite

204 section's centroid than the steel section's flanges, if the plates are wide enough, the additional

205 flexural capacity achieved may even exceed that of the non-retrofitted section.

206 Figure 2 illustrates the cage method as it is implemented in this work. In the vertical direction,

207 the steel cage formed consists of 4 elements of angular section installed at the corners of the

208 steel-concrete composite column and 4 steel plates placed at the middle of each column side.

209 The longitudinal elements are connected with horizontal plates every 20 cm along the column's

210 height. The whole cage structure can be prepared in two parts, with each part forming an angle

211 that covers 2 of the 4 column sides. Then, the two parts can be placed on the existing column

212 and welded together along two corners. The cover layer of unreinforced concrete applied

213 afterwards has a minimal effect on the total section's capacity and is taken into account as

214 unconfined concrete. Its effective thickness is considered smaller than the typical cover sizes
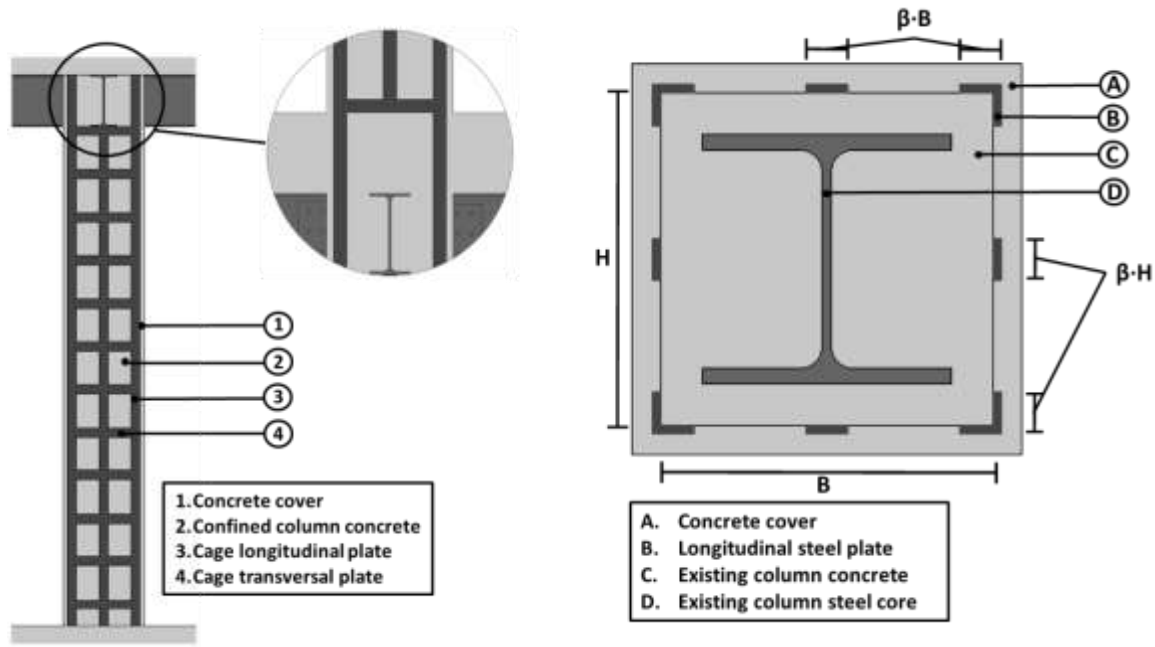
215 applied.

216

217 **Fig. 2.** Column strengthening using concrete-covered steel caging (cage method): side view (left) and cross-
218 section (right) of the retrofitted column.

219 The plates are placed symmetrically at each opposite side of the composite column. Three

220 longitudinal plates are installed at each column side: 2 at the corners and 1 at the middle (Fig.

221 2). Their width is denoted as a percentage β of the initial section's dimensions: if H is the total

222 side width of the unretrofitted section, then the width of each cage plate installed is βH (Fig. 2).

223 A common ratio β is applied to all sides of a column, thus, unless the initial column has a square

224 section, the longitudinal plates installed at perpendicular column sides do not have the same

225 width. Taking also into consideration that a fixed concrete cover of 5 cm around the cage is

226 assumed for all analysis cases, 2 parameters suffice to define the concrete-covered steel cage

8

227 applied to a composite column: (a) the common ratio β of the installed longitudinal plates'
228 width over the width of the corresponding column sides and (b) the common thickness t_c of all
229 plates installed at the column. In this work, a set of 18 pairs of ratios β and thicknesses t_c
230 provides the available discrete options, from which a cage can be selected for retrofitting a
231 column or group of columns. The first option to avoid retrofitting specific columns is available
232 in this dataset as well (β=0). The second option is a very light retrofit solution (t_c=5 mm, β=5%).
233 The last option corresponds to full steel jacketing of the column (β=33.33%, *i.e.* the percentage
234 of each column's side covered by the longitudinal steel plates is 3β=100%). Plates with
235 thicknesses of 10 mm, 15 mm, 20 mm, 25 mm and 30 mm are combined with ratios β of 10%,
236 20% and 30% to form the remaining pairs of available options.

237 **2.2. Overall system retrofit – Adding new members (bracings method)**

238 There is an upper limit of improvement and cost-effectiveness that can be attained by
239 retrofitting existing elements. Therefore, a second retrofit approach has been developed,
240 according to which the structural upgrade of a non-code-conforming building is achieved by
241 strategically introducing additional members, in order to affect the overall seismic performance
242 of the structural system. Such additional elements could be bracings, shear walls, dampers and
243 base isolators, which receive a large amount of the seismic energy during an earthquake,
244 mitigating this way the damage of the actual building. A common characteristic of the
245 aforementioned extra elements is that, when installed, they all affect significantly the building's
246 fundamental period and, therefore, its behaviour under seismic excitation.

247 The third retrofit method investigated in the present paper is the installation of steel bracings in
248 predefined bays of the building. When bracings are added to a moment resisting frame, its
249 overall stiffness is significantly increased, while its fundamental period and the consequent
250 ductility demands are reduced. Moreover, due to the steel material's high ductility, the bracings
251 are able to reach large tensile deformations before failure, absorbing, this way, an adequate
252 amount of seismic energy. This retrofit method is particularly suitable for relatively flexible
253 structural systems, such as deficient moment resisting frames. However, it could have an
254 adverse effect in buildings with increased stiffness, as further stiffness increase might result in
255 amplification of the internal forces at the columns, resulting in early failure.

256 In this work, the technique used to install bracings at bays of reinforced concrete frames is

257 adopted (Fig. 3). Specifically, in order to make a steel surface available, on which connections

258 can be effectively realized, a portal frame of structural steel is installed at the building's bay

259 considered. This steel frame also protects the beam-column joints from failure due to excessive

260 concentrated load applied by the bracings. The frame consists of members with U-shaped steel

261 sections, with which the existing reinforced concrete columns and beam of the bay are

262 enveloped. These steel members can either have standard UPN sections or custom-made

263 sections; usually the second option is preferred to ensure that the steel sections fit to the shapes

264 of the existing columns and beam. The added steel elements are connected to the existing

265 reinforced concrete members using dowels. Then, connection plates are welded on the webs of

266 the U-shaped elements (or the plates are pre-welded on the webs at the supplier's facilities) and

267 the bracings are bolted to the plates. In buildings with steel-concrete composite columns and

268 steel beams, the top part of the installed steel frame can be defined by the existing steel beam,

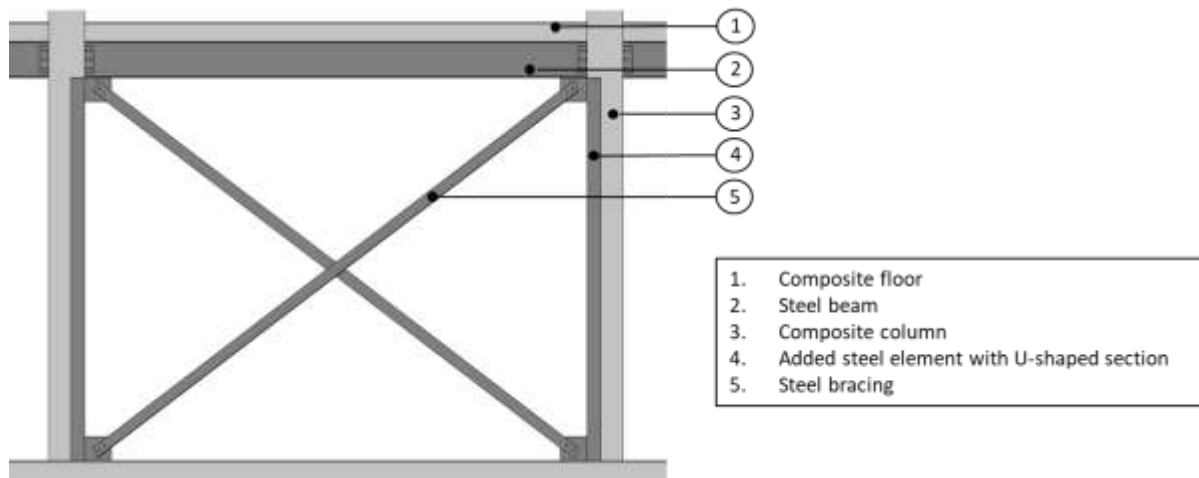269 without adding a new steel member.

270

271 **Fig. 3.** Structural system upgrade using steel bracing (bracings method): side view of bay with installed bracings.

272 In total, 17 standard L-shaped sections comprise the set of available options for bracings.

273 Although this type of bracings is susceptible to flexural buckling, L-shaped sections are often

274 preferred in engineering practice for their reduced cost compared to buckling-restrained braces

275 and ease in assembly. In contrast to hollow sections, which require the installation of a cap plate

276 to weld an additional vertical plate to realize the connection, the web of L-sections plays the

277 role of the connection plate. Additionally, when L-sections fail in buckling due to compressive

278 axial force, they develop significant deformations, absorbing this way an amount of the seismic

279 energy and preventing the occurrence of further damage at the beam-column joint. Each bracing

280 consists of a pair of L-sections, which can be connected at middle length to reduce the effective

281 length for buckling to half their total length. In the investigation performed herein, such a

282 connection is conservatively not considered. Moreover, because L-sections fail in compression

283 relatively early, the bracings' connections with the frame remain practically undamaged. Even

284 though buckling is generally regarded as an unwanted type of failure, the repair cost of a

285 retrofitted building is significantly reduced, when only the bracings need to be replaced instead

286 of the bracings together with the connection plates due to failure of both components. Special

287 attention needs to be paid to the bottom end of the columns, on which the bracings are installed,

288 due to the large concentrated shear force applied. The steel core of the composite columns of

289 the buildings simulated in this work was found to suffice, in order to receive the full amount of

290 shear force. The same does not apply for analogous applications in reinforced concrete

291 structures, where the shear force needs to be received almost entirely by stirrups (or diagonal

292 reinforcement), therefore it is a potential location where shear failure might occur. Note that

293 the database with L-shaped sections additionally includes a 'zero' option (no bracing section),

294 which actually offers the optimizer the choice to deactivate bracings.

295 ## 3. Structural Modelling and Analyses

296 All structural simulations are performed using OpenSEES [43]. The particular structural

297 analysis software provides the capability of handling various material types in a single member

298 section and, therefore, can simulate effectively the composite columns without and with retrofit

299 measures installed. Hence, the existing steel beams and the steel core of the composite columns,

300 as well as the installed bracings and the steel components of the concrete-covered steel cage are

301 assumed to consist of the same quality of structural steel S235 and simulated using the bilinear

302 material model 'Steel01'. Particular attention is paid so that the final retrofitted building designs

303 do not exceed the ultimate strain of steel or the critical stress for buckling, as otherwise

304 overestimated element capacity would be considered. The 'Concrete01' material model is

305 employed for the simulation of all concrete regions in a building, *i.e.* of both the existing

306 composite columns' concrete and that of the retrofit sections. This model is implemented with

307 no tensile capacity, while its compressive strength is taken as 20MPa and the cracking and

308 crashing strains as 2‰ and 3.5‰, respectively. The longitudinal and transverse reinforcement

309 bars of the existing composite columns and of the reinforced concrete jacket are modelled using

310 the 'ReinforcingSteel' material type. This model differs from 'Steel01' in its post-yield branch,

311 as it consists of a horizontal plateau and a hardening-softening part. Its yield stress is taken as

312 500MPa and its ultimate strain as 20%.

313 The non-retrofitted and retrofitted columns, the beams and the bracings are simulated using

314 distributed plasticity (fiber section) elements of OpenSEES. In particular, columns and beams

315 are modelled as 'nonlinearBeamColumn' elements in the x-direction (i.e. parallel to the major

316 axes of all columns' HEB steel cores) and 'beamWithHinges' in the y-direction, while 'truss'

317 elements are used for bracings [43]. The composite slabs are considered to perform as rigid

318 diaphragms at the horizontal plane. The stiffness required for the diaphragm behaviour of the

319 floor in the direction that is orthogonal to the corrugations is provided by secondary beams. The

320 slabs' effect on the structural performance is taken into account by defining a 'rigidDiaphragm'

321 [43] for each storey, while their loads are transferred as distributed loads on the beams. All

322 beam-column connections are considered to be able to fully transfer the loads and moments

323 they receive in the global x-direction and operate as simple supports in the global y-direction.

324 Column base connections are modelled as fixed supports. It is also worth mentioning the

325 assumption made that, in the analysed structural models, an effective connection between the

326 elevator/staircase reinforced concrete core wall that might be present in the 4-storey and 6-

327 storey buildings and the lateral resisting system is not implemented, *i.e.* the core is detached

328 from the lateral resisting system. In the case of an effective connection, the contribution of the

329 core to the overall building stiffness could be significant with a consequent substantial effect

330 on the retrofit optimization results, depending also on the core's position within the building's

331 floor plan.

332 Three types of structural analyses are performed in this work using OpenSEES for any

333 candidate optimum structural design assessed. Initially, a linear static analysis under

334 gravitational loads takes place, in order to apply the gravitational loads on the structural model,

335 as well as to obtain analysis results needed for the initial capacity checks of structural elements.

336 Once the gravitational loads are applied and the total mass of each storey is defined, two

337 eigenvalue analyses are performed (one for each horizontal direction x and y), in order to

338 determine the maximum fundamental period of the building and define parameters required for

339 the next analyses. Finally, two nonlinear static analyses under horizontal loads (pushover

340 analyses) are performed (one for each horizontal direction x and y) up to a targeted top

341 displacement, in order to assess the seismic performance of the building under evaluation.

4. Structural Optimization

343 The Evolution Strategies (ES) [44] optimization algorithm is utilized for the computational

344 investigation performed in the present work. The particular algorithm, which imitates the

345 evolution of a species in time, is a well-established derivative-free optimization method

346 particularly suitable for engineering problems. The optimization procedure employed herein is

347 an adjusted version of the one developed in [40]. In this section, a description is provided for

348 this adjusted implementation, which is tailored to the needs of the optimization problems dealt

349 with in the next section.

4.1. Design variables

351 The basic idea in each case study of the present paper is to initially consider an existing, possibly

352 under-designed moment-resisting frame with specific steel-concrete composite columns and

353 steel beams. Hence, the initial design of the studied building is fixed and the purpose of the

354 optimization procedure is to determine (if needed) an optimal retrofit solution using the

355 approaches presented in section 2. In the sequel of this work, the term 'design' is used to refer

356 to a retrofit solution.

357 The design variables are the parameters, the values of which are altered during the search for

358 the optimum solution. In this paper, the design variables fully control the retrofit solution of a

359 candidate optimum design as described in section 2. Specifically, for the jacket method and for

360 each column-group defined, a design variable is specified to control the jacket's concrete

361 thickness and reinforcing bar size; for the cage method and for each column-group defined, a

362 design variable is specified to control the width and thickness of the steel plates installed at

363 column sides; for the bracings method, two design variables are specified to control the L-

364 shaped sections of bracings installed along directions x and y of the building.

365 The optimization problems handled in the present study are of discrete type: the search space

366 of candidate optimum solutions is defined through the options for design variable values, which

367 are not taken from a continuous range, but from a set of specific (discrete) available sizes of

368 retrofit components (the options for design variable values are defined in section 2). In

369   engineering practice, standardization of dimensions and formation of respective discrete
370   databases of available options is essential, as production of structural components with a limited
371   number of sizes can speed up construction and reduce costs. Optimization runs using a
372   continuous search space would yield retrofit solutions with impractical, non-standard section
373   dimensions. Therefore, for any candidate optimum solution considered herein, each design
374   variable actually takes an integer value (identification number), which corresponds to a
375   particular discrete option provided in the respective database.

376   **4.2. Objective function**

377   The objective of the optimization process is to minimize the total cost of structural materials
378   required to retrofit the building under consideration. The cost of materials for existing elements
379   of the building in its initial (non-retrofitted) state are not taken into account in the employed
380   objective function. Hence, the total cost of structural materials added to retrofit the building can
381   be calculated as the sum of the costs of extra steel and concrete installed when any of the 3
382   retrofit approaches of section 2 is applied. Nevertheless, the calculation of the materials costs
383   in monetary units implies that a 'subjective' final optimization result will be obtained that will
384   depend on the average material prices, which vary at different locations and typically fluctuate
385   with time, i.e. the optimality of a design identified by the optimization procedure will always
386   be linked with a specific location and a certain period of time.

387   A way to improve the 'objectivity' of the utilized optimization procedure is to employ the Cost
388   Ratio CR introduced in [40]. This is defined as $CR=C_c/C_s$, where C_s and C_c are average total
389   unit costs for steel and concrete, respectively. As the cost of concrete is typically calculated
390   based on its volume and that of steel based on its mass, the unit costs C_s and C_c can be specified
391   in €/tn and €/m^3, respectively, thus CR is given in tn/m^3. The cost ratio may also vary with time
392   and location, however other factors, such as a general increase in prices due to inflation or
393   fluctuation in currency exchange rates, are expected to have a small or even no effect on the
394   value of CR. Hence, the cost ratio CR seems to be a more robust choice to link the costs of steel
395   and concrete in an objective function, rather than explicitly using the unit costs C_s and C_c. In
396   this work, a cost ratio of 1.2% (tn/m^3) is adopted for all optimization runs performed, which
397   indicates the availability of 'cheap' concrete and 'expensive' steel, as is typically the case in
398   Cyprus.

399 Following the definition of the cost ratio *CR*, the objective function employed in the

400 optimization procedure measures the total equivalent steel mass of retrofit materials M_s^{tot} (tn

401 of steel) in the structure and can be written as:

402 $$M_s^{tot} = M_S + CR \cdot V_C,\tag{1}$$

403 where M_S and V_C are the total steel mass (tn) and concrete volume (m^3), respectively, of retrofit

404 materials used in the structure. Hence, the objective function is expressed as the sum of the

405 actual steel mass and the converted concrete mass of installed materials to retrofit the building

406 under consideration.

407 **4.3. Constraints**

408 In a structural optimization problem, constraint functions are evaluated using the results of

409 structural analyses for each candidate optimum design, in order to assess the overall

410 performance of the building, as well as of its individual structural components, with respect to

411 predefined criteria. The constraints imposed in the framework of the optimization procedure in

412 this work are:

413 (a) The retrofitted structure for any candidate optimum design is required to satisfy the

414 capacity criteria defined in Eurocode 3 [45] for pure steel members and Eurocode 4 [42]

415 for steel-concrete composite members. The provisions of Eurocode 4 regarding the

416 capacity in axial force, shear force, bending moment, combined axial force and biaxial

417 bending moment and the respective types of local and global buckling are evaluated to

418 check the composite columns. The pure steel beams are checked for their capacity in shear

419 force, bending moment and their interaction, as well as the respective types of local and

420 global buckling according to the provisions of Eurocode 3. The aforementioned checks are

421 performed based on the results of the initial linear static analysis of the structure. Note that,

422 although bracings are pure steel members, they are not checked with respect to the

423 provisions of Eurocode 3. The bracings' contribution to the load transferring mechanism

424 of the structure is actually activated under seismic action, therefore bracing sections are

425 determined based on the global structural system performance and not on local

426 member/section capacity criteria.

427 (b) The overall structural performance of the retrofitted structure under horizontal loading is

428 assessed in accordance with provisions of ASCE/SEI 41-06 [46]. In particular, two

429 displacement-controlled pushover analyses (in horizontal directions x and y) up to the

430 targeted top displacement specified in FEMA-440 [47] are performed for each candidate

431 optimum design. The maximum interstorey drift is used as an overall structural

432 performance indicator. Its maximum value is retrieved from Table C1-3 of ASCE/SEI 41-

433 06 [46] for the collapse prevention limit state. The limit values provided therein are 4% of

434 the storey height for reinforced concrete frames and 5% for steel frames. As there is no

435 provision for steel-concrete composite frames, the 4% limit for reinforced concrete frames

436 is selected herein as a conservative requirement.

437 (c) A limit on the maximum fundamental period of the retrofitted building is imposed using

438 the formula of Goel and Chopra [48] for steel moment-resisting frames (again, there is no

439 respective information specifically for steel-concrete composite frames).

440 All aforementioned constraints need to be satisfied by any candidate optimum design to be

441 considered feasible. Violation of at least one requirement renders the assessed design infeasible

442 and the optimization algorithm adds a penalty to the objective function value. Specifically, the

443 total equivalent steel mass for retrofit materials of an infeasible design is increased by the

444 respective mass of a building with the same geometrical characteristics, which is retrofitted

445 with the largest available sections in the utilized databases, rounded up to 50 tons. It should be

446 noted that, while the local member/section capacity checks of Eurocodes 3 and 4 are used as

447 feasibility criteria using the linear static analysis results, the same does not apply to the

448 performance of structural components under seismic action. Hence, individual components are

449 allowed to fail during both pushover analyses performed, provided that such local failures do

450 not trigger partial or full collapse of the analysed building.

451 **5. Optimization Results**

452 In this section, the cost-effectiveness of the retrofit methods described in section 2 is assessed

453 using the presented optimization procedure. For this purpose, three buildings are assessed,

454 which have the same 5-by-5-bay symmetric floor plan, but a different number of storeys: a 2-

455 storey, a 4-storey and a 6-storey building. The 6-storey structure is illustrated in Fig. 4. For all

456 three buildings, the span of each beam is 6m, calculated as the distance between the centroids

457 of the two columns, to which the beam is attached. As regards columns, all have a height of

458 3.5m and are considered to have the same orientation, with their HEB steel cores' major axes

459   being parallel to the global *x*-direction. Hence, a global 'major axis' and a global 'minor axis'

460   (parallel to the global *x*- and *y*-directions, respectively) are defined for each storey, as well as

461   for the whole building as a system.

462   The first step of the assessment procedure followed herein is to design the three structures in a

463   way that all requirements outlined in subsection 4.3 are satisfied using the smallest possible

464   member sizes (reference buildings). A single size of HEB steel core is used for all composite

465   columns of each building. In preliminary analyses it was noticed that the beam sections required

466   for the gravitational loads generally suffice when the buildings are evaluated for horizontal

467   loads. Hence, it remains to identify the columns' smallest possible HEB size for each reference

468   building, which renders the design of the 3 code-conforming buildings a simple trial-and-error

469   procedure: starting from a small HEB steel core size, it is increased one standard size at a time

470   until a design is determined that satisfies all criteria of subsection 4.3. Thus, composite columns

471   with HE550B steel cores are required for the 2-storey reference building and HE800B for the

472   4-storey reference building, both designed as Moment-Resisting Frames (MRFs). For the 6-

473   storey building, the largest available column section does not suffice, therefore it is designed as

474   a braced frame: the minimum bracing size is used at the corner bays (as indicated in Fig. 4) and

475   the above mentioned trial-and-error procedure is followed. The design identified for the 6-

476   storey reference building with this procedure has composite columns with HE320B steel cores

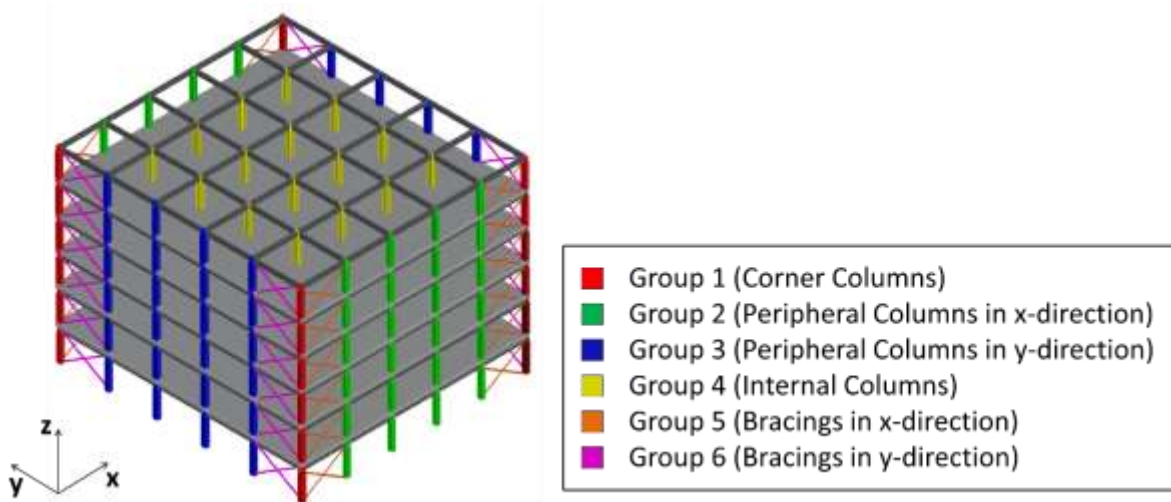477   and L90×90×7 steel bracings at the corner bays.

478

479   **Fig. 4.** Member groups illustrated on the 6-storey building (top slab removed for visualization purposes).

480 Using smaller steel core sizes for the composite columns than the ones determined for each
481 reference building above, a total number of 30 under-designed buildings are generated. More
482 specifically, sections down to HE180B are used for the 2-storey MRF (13 cases of deficient
483 buildings), sections down to HE280B for the 4-storey MRF (12 cases) and sections from
484 HE220B to HE300B for the 6-storey braced frame (5 cases). Note that, as in the case of the 6-
485 storey reference building, all under-designed and unretrofitted 6-storey buildings have the
486 minimum L90×90×7 steel bracings installed at the corner bays. In all 30 under-designed cases,
487 the buildings are retrofitted using each of the three methods described in section 2 in the
488 framework of the utilized optimization procedure. To facilitate the optimization process,
489 columns are organized into 4 groups according to their location in the floor plan (column groups
490 1-4 in Fig. 4): (1) corner, (2) peripheral at the sides parallel to global x-direction, (3) peripheral
491 at the sides parallel to global y-direction and (4) internal. Hence, the columns of each group
492 have a constant size along the height of the building. Two additional groups are defined for the
493 bracings (bracings groups 5, 6 in Fig. 4): (5) at the corner bays of the sides parallel to global x-
494 direction and (6) at the corner bays of the sides parallel to global y-direction. One discrete
495 design variable controls the retrofit choice for each of these 6 member-groups. As already
496 mentioned in section 2, 'zero' options are included in all utilized databases and are available to
497 be chosen as design variable values. This allows the optimization procedure, in its effort to
498 identify the most cost-effective retrofit solution, to activate or deactivate the 2 column
499 strengthening approaches (jacket and cage methods) for any of the 4 column-groups and the
500 bracings along any of directions x and y.

501 Four different retrofit optimization runs are performed for each of the under-designed buildings
502 defined. In each of the first two optimization runs, only one of the column-strengthening
503 methods is enabled: the jacket method is applied in the first run and the cage method in the
504 second run. For the remaining two optimization runs, the bracings method is enabled in
505 combination each time with one of the two aforementioned column strengthening methods. In
506 the case of the 6-storey braced frame, the bracings chosen to retrofit the building are assumed
507 to replace the ones initially installed (if different). A total number of 120 retrofit optimization
508 runs were performed, the results of which are presented in Tables 1, 2 and 3 for the 2-storey, 4-
509 storey and 6-storey buildings, respectively.

18

A macroscopic conclusion drawn from the results of these tables is that the optimal retrofit approach is decisively affected by: (a) how much under-designed a building is compared to the corresponding feasible non-retrofitted building designed initially and (b) the type of structural system of the building (MRF or braced frame). These factors are related to the fundamental period of the building that seems to play an important role in the process of identifying a cost-effective retrofit solution. Indeed, certain building designs exhibited maximum interstorey drifts that were considerably less than the imposed limit of 4% of the storey height, however they also had rather high fundamental periods that rendered them unacceptable. As a result of increasing the structural system's stiffness to address the high fundamental period problem, the maximum recorded interstorey drifts were further reduced (actually, they do not exceed the value of 2.2% of the storey height for all optimized retrofit solutions in this paper). Hence, a designer could use the eigenvalue analysis results to have a strong indication of design feasibility or infeasibility and avoid the need to also perform pushover analyses for designs that would be proven infeasible after all. Clearly, for a retrofit solution with acceptable fundamental period, subsequent pushover analysis results are required to formally check interstorey drifts.

Table 1. Optimized retrofit solutions for under-designed 2-storey buildings.

REINFORCED CONCRETE JACKETS

Column sections (steel core)	Column group 1	Column group 2	Column group 3	Column group 4	Total retrofit demand (equivalent kg steel)
HE180B	d=12mm, t_j=25cm	d=12mm, t_j=15cm	Not required	d=12mm, t_j=20cm	11,460
HE200B	Not required	Not required	Not required	d=12mm, t_j=25cm	8,983
HE220B	Not required	Not required	Not required	d=12mm, t_j=25cm	9,252
HE240B	Not required	Not required	Not required	d=12mm, t_j=25cm	9,521
HE260B	Not required	Not required	Not required	d=12mm, t_j=20cm	7,612
HE280B	Not required	Not required	Not required	d=12mm, t_j=20cm	7,827
HE300B	d=12mm, t_j=10cm	Not required	Not required	d=12mm, t_j=15cm	7,096
HE320B	Not required	Not required	Not required	d=12mm, t_j=15cm	6,107
HE340B	Not required	Not required	Not required	d=12mm, t_j=15cm	6,187
HE360B	Not required	Not required	d=12mm, t_j=20cm	Not required	4,182
HE400B	Not required	Not required	d=12mm, t_j=15cm	Not required	3,215
HE450B	Not required	Not required	d=12mm, t_j=10cm	Not required	2,341
HE500B	d=12mm, t_j=10cm	Not required	Not required	Not required	1,204
≥ HE550B	-	-	-	-	Not required

CONCRETE-COVERED STEEL CAGES

Column sections (steel core)	Column group 1	Column group 2	Column group 3	Column group 4	Total retrofit demand (equivalent kg steel)
HE180B-HE360B	-	-	-	-	Infeasible
HE400B	t_c=20mm, β=10%	t_c=40mm, β=33,3%	t_c=40mm, β=33,3%	t_c=40mm, β=33,3%	106,946
HE450B	Not required	Not required	Not required	t_c=25mm, β=10%	11,003
HE500B	Not required	Not required	Not required	t_c=10mm, β=10%	5,198
≥ HE550B	-	-	-	-	Not required

STEEL BRACINGS

Column sections (steel core)	Bracings group 5	Bracings group 6	Total retrofit demand (equivalent kg steel)

| HE180B-HE500B | L 90×90×7 | L 90×90×7 | 3,039 - 3,756 |
| **≥ HE550B** | - | - | Not required |

526

Table 2. Optimized retrofit solutions for under-designed 4-storey buildings.

REINFORCED CONCRETE JACKETS

Column sections (steel core)	Column group 1	Column group 2	Column group 3	Column group 4	Total retrofit demand (equivalent kg steel)
HE280B	d=12mm, t_j=25cm	d=12mm, t_j=25cm	d=20mm, t_j=25cm	d=12mm, t_j=25cm	48,090
HE300B	d=25mm, t_j=25cm	d=12mm, t_j=25cm	d=12mm, t_j=20cm	d=12mm, t_j=25cm	46,844
HE320B	d=12mm, t_j=10cm	d=12mm, t_j=20cm	d=12mm, t_j=25cm	d=12mm, t_j=25cm	41,700
HE340B	d=12mm, t_j=10cm	Not required	d=12mm, t_j=25cm	d=12mm, t_j=25cm	33,981
HE360B	Not required	Not required	d=12mm, t_j=25cm	d=12mm, t_j=25cm	32,190
HE400B	d=12mm, t_j=20cm	Not required	Not required	d=12mm, t_j=25cm	26,288
HE450B	d=12mm, t_j=15cm	d=12mm, t_j=10cm	Not required	d=12mm, t_j=20cm	25,695
HE500B	Not required	Not required	d=20mm, t_j=15cm	d=12mm, t_j=15cm	23,326
HE550B	d=25mm, t_j=25cm	Not required	Not required	d=12mm, t_j=10cm	18,563
HE600B	d=12mm, t_j=10cm	Not required	Not required	d=12mm, t_j=10cm	12,713
HE650B	Not required	Not required	d=25mm, t_j=10cm	Not required	10,534
HE700B	Not required	Not required	d=12mm, t_j=10cm	Not required	5,354
≥ HE800B	-	-	-	-	Not required

CONCRETE-COVERED STEEL CAGES

Column sections (steel core)	Column group 1	Column group 2	Column group 3	Column group 4	Total retrofit demand (equivalent kg steel)
HE280B-HE600B	-	-	-	-	Infeasible
HE650B	t_c=10mm, β=10%	t_c=40mm, β=33,3%	t_c=40mm, β=33,3%	t_c=40mm, β=33,3%	276,988
HE700B	Not required	Not required	Not required	t_c=20mm, β=10%	23,841
≥ HE800B	-	-	-	-	Not required

STEEL BRACINGS

Column sections (steel core)	Bracings group 5	Bracings group 6	Total retrofit demand (equivalent kg steel)
HE280B-HE700B	L 90×90×7	L 90×90×7	6,730 - 8,164
≥ HE800B	-	-	Not required

527

Table 3. Optimized retrofit solutions for under-designed 6-storey buildings.

REINFORCED CONCRETE JACKETS

Column sections (steel core)	Column group 1	Column group 2	Column group 3	Column group 4	Total retrofit demand (equivalent kg steel)
HE220B	d=20mm, t_j=10cm	Not required	d=12mm, t_j=10cm	d=12mm, t_j=10cm	22,328
HE240B	d=12mm, t_j=10cm	Not required	d=12mm, t_j=10cm	d=12mm, t_j=10cm	20,771
HE260B	Not required	Not required	Not required	d=12mm, t_j=10cm	12,192
HE280B	Not required	Not required	d=12mm, t_j=10cm	Not required	6,257
HE300B	d=12mm, t_j=10cm	Not required	Not required	Not required	3,209
≥ HE320B	-	-	-	-	Not required

CONCRETE-COVERED STEEL CAGES

Column sections (steel core)	Column group 1	Column group 2	Column group 3	Column group 4	Total retrofit demand (equivalent kg steel)
HE220B	-	-	-	-	Infeasible
HE240B	t_c=25mm, β=10%	t_c=5mm, β=5%	t_c=30mm, β=10%	t_c=25mm, β=10%	40,868
HE260B	t_c=10mm, β=10%	t_c=15mm, β=10%	t_c=15mm, β=10%	t_c=5mm, β=5%	21,238
HE280B	t_c=5mm, β=5%	Not required	t_c=5mm, β=5%	t_c=5mm, β=5%	5,344
HE300B	Not required	Not required	t_c=5mm, β=5%	Not required	1,614
≥ HE320B	-	-	-	-	Not required

STEEL BRACINGS

Column sections (steel core)	Bracings group 5	Bracings group 6	Total retrofit demand (equivalent kg steel)
HE220B-HE240B	-	-	Infeasible
HE260B-HE300B	L 120×80×8	L 120×80×8	11,524 - 11,907
≥ HE320B	-	-	Not required

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528 In the remainder of this section, specific remarks are made on the effectiveness of each the 3
529 retrofit approaches assessed based on the obtained optimization results.

530 **5.1. Cage Method**

531 The cage method is the least invasive of the retrofit approaches assessed in the present work,
532 as its application results only in a small increase of the section areas of the retrofitted columns.
533 Therefore, the method's effectiveness in increasing the capacity of columns is limited. Hence,
534 the results obtained herein show that this method is really effective in increasing the columns'
535 capacity and stiffness and improving a building's overall performance when only limited
536 strengthening is required. However, for a well under-designed building with rather weak
537 columns, there is an abrupt increase in the total retrofit cost, because the distances of the
538 installed steel cages from the columns' centroids are fixed, unlike in the case of reinforced
539 concrete jackets, in which jacket thicknesses can be increased to move steel reinforcing bars
540 away from the columns' centroids.

541 Indeed, as regards the MRFs of the 2-storey and 4-storey buildings, it can be seen in Tables 1
542 and 2 that feasible retrofit solutions can be determined using the cage method for a rather limited
543 number of cases, in which deficient columns are only a little weaker than the ones of the
544 corresponding code-conforming reference buildings. For under-designed buildings with even
545 weaker columns than the ones successfully retrofitted, no feasible retrofit solutions can be
546 identified using the cage method. Notice that, in the weakest of the retrofitted 2-storey and 4-
547 storey buildings, the total retrofit cost is so high that it renders the cage method practically
548 unacceptable, even if architectural constraints discourage the application of an alternative, more
549 invasive retrofit method. Hence, the cage method appears to be inefficient in retrofitting the 2-
550 storey and 4-storey MRFs, as it struggles to find costly column caging designs with wide and
551 thick steel plates (as indicated by the values of t_c and β in Tables 1 and 2), which do not succeed
552 in satisfying the specified design requirements in most of the cases studied.

553 As regards the 6-storey buildings (Table 3), in the cases of columns with HE300B and HE280B
554 steel cores, the identified optimal retrofit solutions are not only feasible, but are also the most
555 cost-effective ones, outperforming the other two retrofit methods. As already mentioned, all
556 under-designed 6-storey structures are braced frames, with the installed bracings resulting in
557 significantly increased total stiffness and, consequently, in reduced fundamental period for

21

558 these frames. Even though all deficient 6-storey buildings violate the maximum admissible

559 interstorey drift constraint as well, the fundamental period limit is barely exceeded. Hence, a

560 limited strengthening of selected composite columns (with minimal steel caging using t_c=5 mm,

561 β=5%) is really effective and beneficial for both interstorey drift and fundamental period criteria

562 (in addition to the structural member capacity criteria defined in Eurocode 4), which explains

563 the success of the cage method in providing the most cost-effective designs for the two

564 particular cases mentioned above. Furthermore, this method manages to determine feasible

565 designs for two more under-designed 6-storey structures with even weaker columns, although

566 the other two retrofit methods suggest more economical solutions for these two cases. Finally,

567 it is observed in Table 3 that the increase in the total retrofit cost for decreasing column section

568 size is considerably smoother in the case of the 6-storey buildings than those observed in Tables

569 1 and 2 for the lower buildings.

570 **5.2. Jacket Method**

571 From the results of Tables 1 to 3, it is evident that the jacket method can be effectively applied

572 to a significantly wider range of under-designed buildings than the cage method. In fact, the

573 jacket method manages to provide feasible retrofit solutions for all deficient column cases

574 considered in this section. Moreover, the increase in the total retrofit cost for decreasing column

575 section size is generally much smoother than in the cage method. In all studied cases concerning

576 the seismic retrofit of pure MRFs (*i.e.* the 2-storey and 4-storey structures), the jacket method

577 provides more cost-effective retrofit solutions than the cage-method. It is indicative that the

578 retrofit of the 2-storey building with HE180B column steel cores by the jacket method requires

579 about the same total equivalent steel mass as the retrofit of the 2-storey building with HE450B

580 column steel cores using the cage method. The jacket method actually succeeds in providing

581 more cost-effective solutions than both other retrofit approaches in the cases of 2-storey

582 buildings having columns with HE500B or HE450B steel cores, as well as in the case of the 4-

583 storey building having columns with HE700B steel cores.

584 As regards the braced frames of the 6-storey buildings examined, the jacket method outperforms

585 the cage method in all cases studied except for the ones with HE300B and HE280B column

586 steel cores. Note also that the jacket method is actually the only approach that is able to provide

587 a feasible retrofit solution for the 6-storey building with the weakest columns (HE220B steel

588 cores). This is related to the fact that, while the steel bracings increase the overall stiffness of

589 the building, they do not enhance the columns' moment resistance. Hence, in the case of a

590 braced frame with very weak columns, while the structure is capable of receiving the

591 gravitational loads (self-weight and imposed loads due to typical use), the design bending

592 moments developed in the columns at the Ultimate Limit States exceed the columns' resistance.

593 Hence, their sections need to be adequately strengthened, which can only be achieved using the

594 jacket or the cage method.

595 In the cases where the same retrofit solution is identified by the optimizer for two different

596 deficient buildings of the same height, it is observed that the total retrofit mass demanded is

597 higher for the building with larger initial column section size. This is due to the way the selected

598 retrofit option is defined, *i.e.* by specifying only the jacket thickness and the reinforcement

599 diameter. Hence, the actual dimensions of the jacket are related to the respective dimensions of

600 the existing column. This results in larger total jacket concrete volume for larger existing

601 column sections. To reduce the additional retrofit cost for such designs, a finer database with

602 extra, intermediate retrofit options could be provided for the optimizer to choose from.

603 Compared to the cage method, the jacket method offers retrofit solutions that require more

604 space in the floor plan to be applied. If there are relevant architectural constraints to limit the

605 degree of retrofit invasiveness, these could be taken into account through a penalty function

606 increasing the objective function value proportionally to the additional area covered by the

607 retrofitted column sections. Hence, although such constraints are not considered in the present

608 work, there are ways to effectively deal with these, if needed.

609 It is worth noting that, with few exceptions, the optimized retrofit solutions of the jacket method

610 use longitudinal steel reinforcement bars with the minimum diameter available in the respective

611 database (d=12mm). The optimization algorithm seems to prefer jacket designs with this rebar

612 size, because they exploit the jacket concrete as a low-cost means to increase the distance of

613 the steel rebars from the columns' centroids. Hence, a column's capacity and stiffness can be

614 significantly increased with a proportionally smaller increase in the total retrofit cost. To

615 validate the optimality of the jacket designs determined by the optimization procedure, all

616 optimized solutions with at least one column group having jackets with rebar diameter larger

617 than 12mm were further investigated manually. Indeed, for these cases, the most cost-effective

618 feasible retrofits using only rebars with d=12mm were found to have a higher total cost than

23

619   the designs output by the optimizer with larger rebar sizes. Hence, despite the aforementioned

620   advantage of small-size rebars in thick jackets, the optimization algorithm was able to

621   automatically identify certain exceptions, in which more cost-effective retrofit solutions are

622   available using rebars of larger size. As an alternative to jacket designs using rebars with large

623   diameters, a larger number of small-size rebars per side could be utilized, provided that there is

624   adequate space for their installation.

625   Another interesting observation is that, except for the retrofit of 4-storey MRFs with very weak

626   columns, the optimizer exhibits a general tendency not to strengthen all columns in a retrofit

627   design, but to activate jacketing only for certain column groups each time. The jacketed column

628   groups are not the same in every optimization run performed and can be 1, 2, 3 or 4, depending

629   on the features of the particular retrofit problem at hand. This makes it difficult to manually

630   identify the optimized retrofit solution in each case considered and emphasizes the effectiveness

631   and usefulness of the presented optimization procedure.

632   **5.3. Bracings Method**

633   The installation of bracings in a MRF is a particularly effective method to reduce its

634   fundamental period, leading to decreased ductility demands, which are imposed in this work

635   through the targeted top displacement for the pushover analysis. This effect is confirmed by the

636   results of Tables 1 and 2, which show that the optimization algorithm was able to identify

637   feasible retrofit designs for all 2-storey and 4-storey under-designed MRFs using only the

638   smallest available L-section for the bracings at the corner bays of all building sides. When the

639   results of Table 1 (2-storey MRFs) and Table 2 (4-storey MRFs) are examined separately, it

640   can be observed that, although the same bracing section is utilized in all retrofit solutions,

641   deficient buildings with smaller column section sizes require less steel mass to be retrofitted

642   with the bracings method than buildings with larger column section sizes. Indeed, smaller

643   column section dimensions lead to increased need for steel mass due to longer bracings, but

644   also to reduced need for steel mass due to smaller supporting frame dimensions. The savings in

645   steel mass for the supporting frame are larger, which explains the noticeable reduction in the

646   total retrofit cost for decreasing column section size.

647   As regards the under-designed 6-storey buildings (Table 3), optimized retrofit solutions can be

648   provided for 3 cases (with HE300B to HE260B steel cores) by replacing the bracings at all

649 building sides with stronger ones. Again, as explained above, smaller column section sizes lead

650 to reduced total retrofit costs. Nevertheless, for the 2 buildings with the weakest columns

651 (HE240B and HE220B steel cores), the bracings method is unable to provide feasible retrofit

652 solutions. For these 2 cases, feasible and possibly affordable retrofit designs can only be

653 obtained by keeping the initial bracings and strengthening 3 column groups using the jacket

654 method (Table 3).

655 It is of particular interest that none of the identified optimal retrofit designs using the bracings

656 method is a mixed solution, *i.e.* a retrofit design specifying the replacement of bracings

657 combined with the strengthening of columns using either the cage or the jacket method. This is

658 a very convenient outcome, because, unless the use of bracings is prohibited, a designer can

659 manually determine a cost-effective retrofit solution without the need to run a time-demanding

660 optimization procedure. More specifically, once the bays where bracings can be installed are

661 defined, a designer only needs to check the building performance using the available bracings'

662 sections from the smallest size to larger ones until a feasible design is determined. Especially

663 in retrofitting relatively slender buildings, the use of bracings can be very advantageous in the

664 framework of either a manual or an automatic optimization procedure.

665 However, when either bracing sections are increased or bracings at new locations are installed

666 in a structure, special care must be taken for columns, which need to be able to carry

667 concentrated shear forces at the connections with bracings. Despite the fact that the building's

668 fundamental period is reduced when larger or additional bracings are installed leading to

669 reduced ductility demands, deficient columns may fail and cause local collapse during pushover

670 analysis before the structure reaches the targeted top displacement. In such cases, column

671 strengthening methods need to be applied in combination with the bracings method to achieve

672 the desired structural system performance. This need was not encountered in any retrofit case

673 studied in the present paper.

674 **6. Concluding remarks**

675 Today's stock of old structures that are under-designed with respect to current design codes is

676 rather large. Improving the structural behaviour and performance of such structures requires the

677 development and validation of effective retrofit approaches, but also vast budgets for their

678 application in practise. The aim of the present work is to comparatively assess the cost-

effectiveness of a number of seismic retrofit approaches for deficient buildings with steel-concrete composite columns, in order to facilitate the selection of the economically most viable intervention depending each time on the particular case at hand.

Three different seismic retrofit approaches are studied in this paper. Reinforced concrete jacketing and concrete-covered steel caging are two local retrofit approaches that aim in indirectly upgrading structural performance at the global system level through individual column strengthening. The installation of steel bracings at selected bays of a structure is a global retrofit approach that focuses directly on enhancing system resistance by adding new structural elements. These 3 retrofit approaches were compared on a 'fair', objective basis using a specially developed structural optimization procedure to automatically determine the most cost-effective retrofit solution for each case studied, without relying on the capabilities, experience and subjectivity of a particular designer. All assessed retrofit approaches were found to be effective in improving structural performance, but none of the approaches was found to be the most suitable and cost-effective for all cases of deficient buildings considered.

Based on the retrofit solutions obtained from a total number of 120 optimization runs for 30 cases of under-designed 2-storey, 4-storey and 6-storey buildings, the following main conclusions can be drawn regarding the advantages and relative cost-effectiveness of each of the 3 retrofit approaches studied:

- For lightly under-designed buildings (*i.e.* the total interstorey stiffness deficiency is up to about 30% that of the code-conforming reference building), the installation of concrete-covered steel cages at selected or even all composite columns appears to be the most tractable retrofit approach. When limited additional strength and stiffness are required, this approach provides retrofit solutions that are simultaneously the most cost-effective and least aesthetically intervening, improving also the confinement of the existing concrete of the composite columns.

- In the case of higher requirements for additional column capacity and stiffness of under-designed buildings, the installation of reinforced concrete jackets at selected or even all composite columns provides more cost-effective retrofit solutions. This approach exploits the jacket thickness of the retrofitted element, in order to place reinforcing steel bars at large distances from the section's centroid and ensure this way their increased contribution to the column's stiffness and flexural capacity. This is the only approach that managed to provide

26

710 feasible retrofit solutions for all cases studied in this paper, from lightly to overly under-

711 designed buildings (*i.e.* with total interstorey stiffness deficiency even up to 80% that of the

712 code-conforming reference building). Nevertheless, there are technical and practical

713 limitations associated with the selection of jacket thicknesses, therefore there are also

714 restrictions in the applicability and effectiveness of this retrofit approach.

715 • The installation of adequate steel bracings seems to be often necessary in overly under-

716 designed buildings. The weak columns of such non-retrofitted buildings result in low

717 stiffness and rather high, unacceptable fundamental periods. Bracings change the structural

718 system and its behaviour, effectively shift its fundamental period, but do not necessarily

719 alleviate the deficient composite columns. Therefore, if the capacities of the columns do not

720 suffice, bracings need to be combined with one of the two aforementioned column

721 strengthening approaches (jacketing or caging). If the columns' capacities suffice, the

722 installation of adequate bracings without any other additional intervention can decisively

723 improve a composite building's performance and provide the most cost-effective retrofit

724 solution.

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