

CONCRETE FRAME CORNERS IN CIVIL DEFENCE SHELTERS SUBJECTED TO NEGATIVE MOMENT

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Abstract

A research project has been initiated to evaluate a new reinforcement detailing in frame corners in civil defence shelters where all reinforcement bars are spliced within the frame corner. Full-scale frame corners subjected to negative bending moment have been tested and analysed using the non-linear finite element programme DIANA. A concrete material model based on non-linear fracture mechanics to account for cracking and plasticity models for concrete in compression and reinforcement steel was used. The bond-slip relation between reinforcement and surrounding concrete was accounted for by an interface model. Parameters varied in the study are: reinforcement detailing, reinforcement ratio, the effects of the weakness of the construction joint and the interaction between reinforcement and concrete. Tests and finite element analyses support the idea that the new reinforcement detailing is appropriate to use. Finite element analyses showed that the effect on the maximum load capacity of different bond-slip relations and the weakness of the construction joint were negligible. Key words: Non-linear fracture mechanics, finite element analysis, reinforced concrete, frame corners, splicing of reinforcement

1 Introduction

The reinforcement detailing allowed in the present Swedish shelter regulations, Swedish Rescue Services Agency (1994), for the design of frame corners which are subjected to negative moment in civil defence shelters is time consuming and quite difficult to carry out correctly. Consequently, there is a demand for a simpler reinforcement detailing to be used. Therefore, a new design proposal, in which all the reinforcement bars are spliced within the corner area, was worked out, see Fig. 1. The aim of this study is to evaluate the new design proposal and to determine whether it is appropriate to replace the conventional reinforcement detailing with the new kind. The service criterion set is that the new reinforcement detailing must withstand loading at least as well as the conventional detailing so that a safe and ductile structure is obtained.

To examine the effectiveness of the new detailing, a combination of experiments and non-linear finite element analyses (FE analyses) were carried out. The parameters varied in the tests were the reinforcement detailing, the reinforcement ratio, the reinforcement type and the configuration of the reinforcement bars. FE analyses were used to compare the new and conventional reinforcement detailing. Furthermore, the effects of different interaction between reinforcement and concrete, and the weakness of the construction joint, and the influence of the concrete fracture energy were examined using this method.

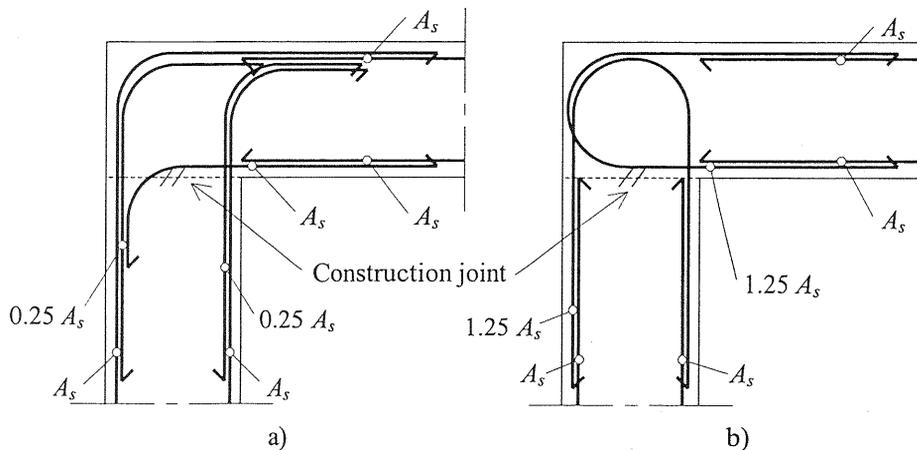


Fig. 1. Detailing of the reinforcement according to: a) the conventional method, and b) the new alternative. A_s denotes the reinforcement area.

2 Experiments

The emphasis of this paper is on non-linear finite element analyses; therefore, only a brief description of the tests is presented. More thorough information about the test series can be found in Plos (1995) and Johansson (1996).

To gain a better understanding of the behaviour of frame corners under loading to failure, two test series, each containing of four full-scale specimens, were carried out. Two specimens were reinforced with the conventional reinforcement detailing and six with the new proposal. Deformed bars of reinforcement type Ks40S (first test series: $f_{yk} = 400$ MPa) or K500 (second test series: $f_{yk} = 500$ MPa) were used and the mechanical reinforcement ratio, ω_s , in the corner region was approximately either 0.16 or 0.04, where ω_s is defined according to equation (1). These values corresponded approximately to the maximum and minimum amount of reinforcement allowed in the Swedish shelter regulations. Concrete quality with a target cylinder compressive strength of 30 MPa was used in all specimens. Fig. 2 shows the dimensions of the specimens and the test set-up.

$$\omega_s = \frac{A_s f_{sy}}{b d f_c} \quad (1)$$

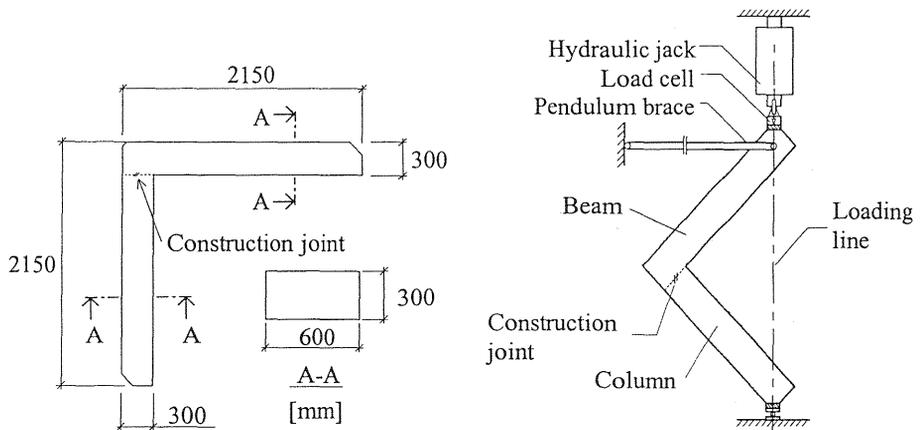


Fig. 2. Dimensions of specimens and test set-up.

3 Non-linear finite element analyses

3.1 General

To study the structural behaviour of the frame corner more thoroughly, the finite element programme DIANA, TNO (1993) used. Hereby, it is possible to evaluate the stresses and deformations of a structure more thoroughly than can be done in an experiment. Accordingly, the use of non-linear finite element analyses will result in a better understanding of the mechanical behaviour in a structure during loading to failure.

The specimens of the second test series were modelled at two different levels of detail. However, in this paper, only the more detailed analyses, of specimens with low reinforcement ratio, are mentioned. For a full description of the analyses carried out, see Johansson (1996).

3.2 Material models

3.2.1 Modelling of the concrete

In the analyses presented here, a fixed non-orthogonal crack model was used. Cracking is initiated by the use of a constant stress cut-off criterion which means that once the maximum principal tensile stress reaches the tensile strength, independent of the other principal stresses, a crack is initiated perpendicular to the principal stress. The orientation of the crack is then stored and the material response perpendicular to the crack is determined by a stress-strain relation for the cracked material volume. Additional cracks may then appear at the same location but their formation is restricted to a minimum angle (here set at 60°) to previous cracks.

To simulate the softening curve of the concrete, a bilinear stress-crack opening relation according to recommendations given in Gylltoft (1983) was used. The fracture energy, G_F , and the tensile strength, f_t , were used to calculate the value of the ultimate crack opening, w_u . In the analyses, the interaction between the reinforcement and the concrete was taken into consideration in the region close to the corner (within 1.0 m of the corner) where cracking of the concrete was expected. Hereby, the crack distribution was given by the analysis and since the smeared cracking of each element represents the development of one real crack, the stress-strain relation of the cracked concrete depends on the length of the finite element. Therefore, in this region, the crack width was divided by the element lengths, $l_{element}$, to determine the softening stress-strain relation used in the analyses. In the regions where perfect bonding between the reinforcement and concrete was assumed, the stress-strain relation for the concrete was determined by dividing the ultimate crack opening by the mean crack distance, s_{rm} (0.2 m), observed in the tests. However, as an approximation to consider the higher stiffness in the structure, due to the

effects of tension stiffening, the gradient of the descending part of the stress-strain curve was halved for the concrete where perfect bond was assumed.

In compression, the response of the concrete was accounted for by an elastic-plastic model. The elastic stress-state was limited by a Drucker-Prager yield surface. Once yielding had occurred, an associated flow rule with isotropic hardening was used. In DIANA, this yield surface is evaluated from the current stress state, the angle of internal friction, ϕ , and the cohesion, c . In accordance with recommendations given in the DIANA manual, the angle of internal friction in concrete was approximated to be $\phi = 30^\circ$. Poisson's ratio was approximated to 0.20.

In Fig. 3 the stiffness used in the analyses for unloaded concrete in tension and compression is shown. When the cracked concrete is unloaded, the secant unloading modulus is used as tangent stiffness so that the strain across the crack is reduced linearly to zero as the stress approaches zero. Thus, in the model used, a crack closes completely when the stress reaches zero. When concrete in compression is unloaded, the initial elastic stiffness is used.

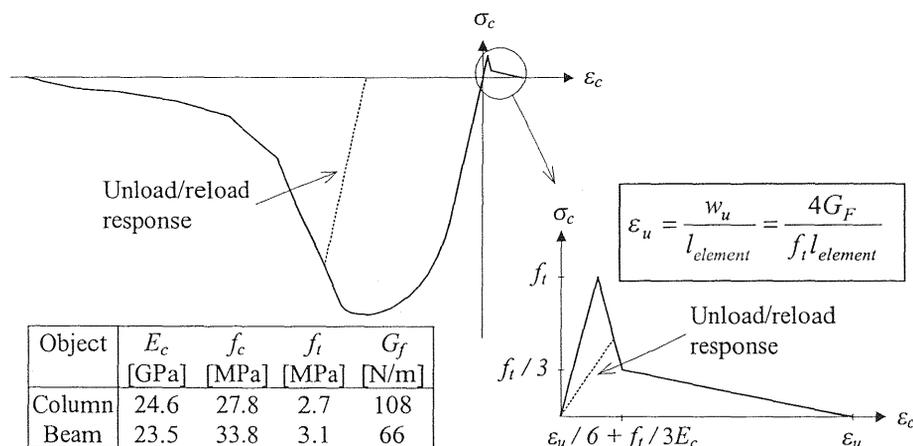


Fig. 3. Stiffness used in the analyses for unloaded concrete in compression and tension ($l_{element} = s_{rm}/2$ when perfect bond is assumed).

3.2.2 Modelling of the reinforcement

The von Mises yield criterion with associated flow and isotropic strain hardening was used to describe the constitutive behaviour of the reinforcement bars. These were modelled with either the DIANA option "embedded reinforcements" or separate steel elements using truss elements. In the embedded reinforcement option, the reinforcement does not have separate degrees of freedom; instead, the strength and stiffness

of the concrete elements are increased in the direction of the embedded reinforcement. With this model, perfect bonding is assumed between the reinforcement and the surrounding material. When the interaction between the reinforcement and the concrete was taken into consideration, the reinforcement bars were modelled by separate elements, using truss elements in combination with interface elements.

In DIANA, interface elements is used to model the bond-slip relation between the reinforcement and the concrete. In these elements the bond stresses only depend on the longitudinal slip; the stresses normal to the bar is described with a stiff linear relation to the deformations in this direction. Thereby, no normal expansion is caused by the slip of a bar and, thus, radial stresses do not arise in the concrete around a bar. Consequently, splitting failures cannot be modelled. Instead, the effect of splitting has to be included in the non-linear bond-slip relation. The bond-slip relation between the reinforcement and the concrete was approximated according to the CEB-FIP Model Code, CEB (1993). To study the effect of the structural behaviour of various bond conditions "good" and "other" bond conditions, assuming "unconfined concrete", were used in the analyses.

3.3 Finite element models

In the FE analyses, two-dimensional plane-stress elements, consisting of four-node quadrilateral elements and three-node triangular elements, were used to model the concrete. The reinforcement bars were modelled using either the embedded reinforcement option available in DIANA or two-node truss elements. Where the reinforcement was modelled by truss elements, four-node interface elements were used to simulate the interaction between the reinforcement and the concrete. Two different FE meshes were used to model the frame corners with the new and the conventional reinforcement detailing. The FE mesh and the modelling of the reinforcement in the frame corner with the new reinforcement detailing are shown in Fig. 4. The material parameters for concrete and reinforcing steel were identical in both models and are listed in Fig. 3.

Furthermore, the effect of the weakness in the construction joint on the structural behaviour of the frame corner was examined. The frame corner specimens were cast with a construction joint, see Fig. 2, and the test results indicated that this joint exhibited a zone of weakness in the material in which cracking first occurred. This was modelled by reducing the tensile strength and the fracture energy of the concrete modelling the construction joint. Thus, the first crack was made to form at the same place as in the tests and since this crack was observed at a load level approximately half of that expected, the tensile strength and fracture energy of the concrete modelling the construction joint was reduced to 50 %. To study the effect of the weakness in the construction joint, a FE

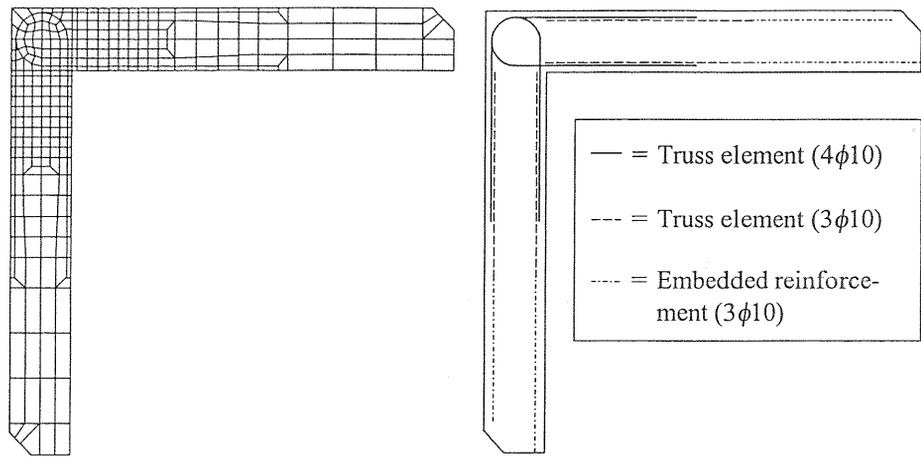


Fig. 4. The finite element mesh and modelling of the reinforcement bars of the frame corner with new reinforcement detailing.

analysis with tensile strength and fracture energy reduced to 90%, compared to that used in the rest of the model, was also carried out for both reinforcement detailings. By using this slight weakness, the first crack was still made to form at the same place as in the tests. Furthermore, the importance of the fracture energy used in the analyses was examined. The fracture energy was halved and an analysis of the specimen with the new reinforcement detailing assuming “good” bond condition was carried out.

3.4 Executions of the analyses

A displacement controlled incremental loading was applied in the FE analyses to model the load of the structure. Two different iteration methods, the Modified Newton-Raphson method and the BFGS secant stiffness method, were tested. It was found that the former method gave a somewhat smoother load-displacement relation than that achieved with the latter. However, the difference in the results obtained when using these methods was negligible and since fewer numerical problems were encountered with the BFGS method, this was the iteration method used. To check the quality of the solution within each increment, a tolerance given as a percentage (usually 0.01 %) of the energy norm was used as the convergence criterion.

It was not possible to predict the behaviour of the frame corner throughout total failure. All the analyses of the frame corner were disrupted due to numerical problems when the ultimate compression strength of the concrete was reached at one integration point in the corner area. At this stage, yielding of the reinforcement bars had occurred and, in some cases, also started to harden.

3.5 Results of the analyses

In the first stage of the analyses, the bond condition corresponding to that of the test specimens was to be determined. Two criteria can be used for this determination: the mean spacing of the major cracks observed in the test specimens and the load-displacement relation obtained in the tests. The former criterion was assumed the better one to use since some factors of uncertainty, such as the fracture energy and the generally stiffer behaviour of the FE analysis, have less effect on it. The crack pattern and load displacement relations of the analyses were compared with those obtained in the tests and the FE analysis assuming "good" bond condition showed the best agreement with the mean crack spacing of 0.2 m observed in the tests. Therefore, a bond-slip relation corresponding to this one was assumed to be the closest to that of the test specimens even though it was somewhat too stiff in comparison with the tests, see Fig. 5.

In Fig. 6, the load-displacement relation of the FE analyses for the frame corner with the new reinforcement detailing (with full and halved fracture energy) is compared with that of the frame corner with the conventional reinforcement detailing.

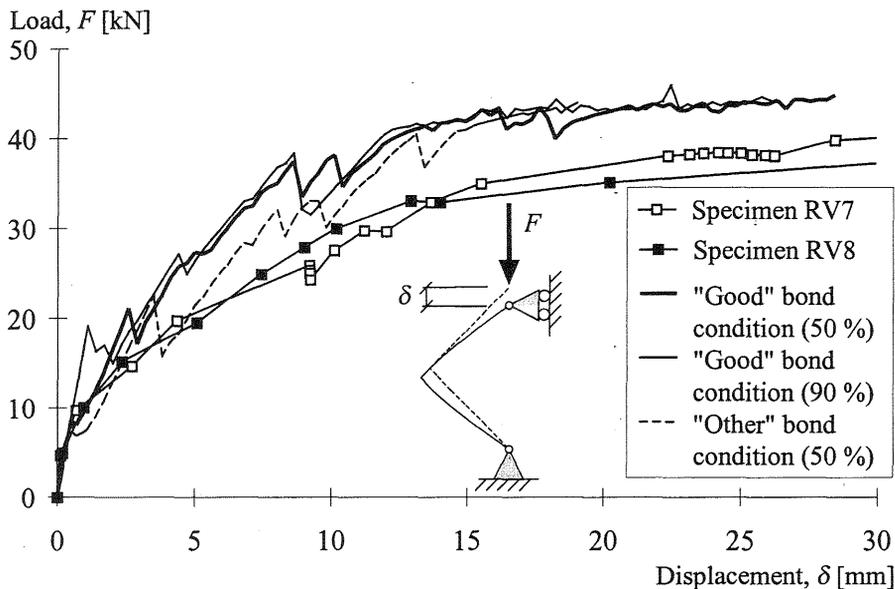


Fig. 5. Load-displacement relations for the tests and FE analyses with the new reinforcement detailing. Values between brackets show the strength of the concrete modelling the construction joint.

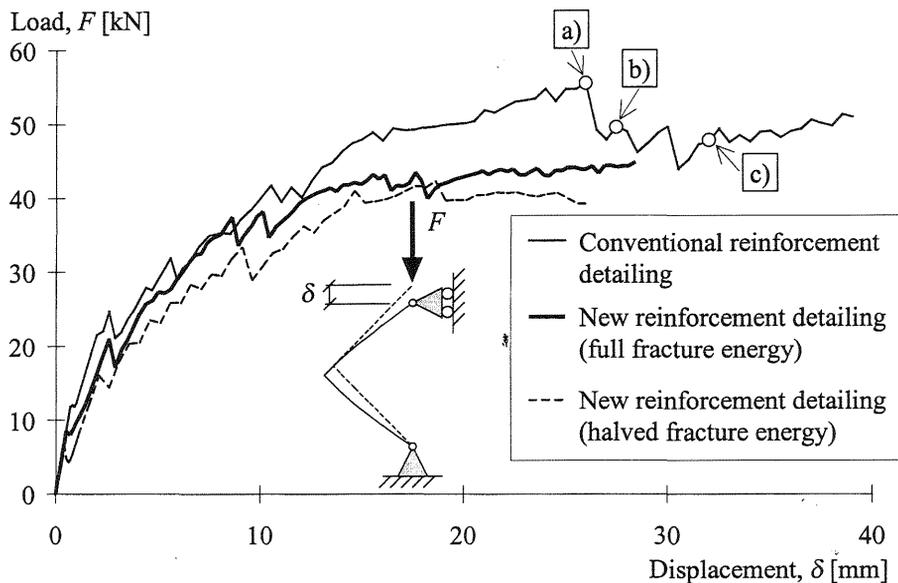


Fig. 6. Comparison of the load-displacement relations for frame corners carried out with new and with conventional reinforcement detailing.

3.6 Discussion

The results of the FE analyses corresponded quite well with the results from the experiments; the behaviour was similar for both the maximum load capacity obtained and the stiffness of the structure. Furthermore, the crack pattern and positioning of the plastic hinges in the detailed analyses corresponded very well with that observed in the test specimens with both the new and conventional reinforcement detailings.

In Fig. 5 it can be seen that the stiffness of the structure is affected by the bond-slip relation up to the point at which the steel reinforcement yields: the higher the stiffness of the bond-slip relation, the higher the stiffness of the structure. However, the effect of the bond-slip relation on the load capacity was negligible. Furthermore, the FE analyses indicate that, if rupture of the reinforcement bars is the cause of failure, then the deformation capacity of the frame corner would be lower for a stiffer bond-slip relation. The influence of the weakness of the construction joint in the frame corner was limited to the initial cracking stage and had negligible effect on the general behaviour of the structure. It is not shown here, but the analyses have shown that these statements hold true also for a frame corner made with the conventional reinforcement detailing.

In Fig. 6 it can be seen that the structural behaviour of the frame corners with the new and conventional detailing was similar up to a load

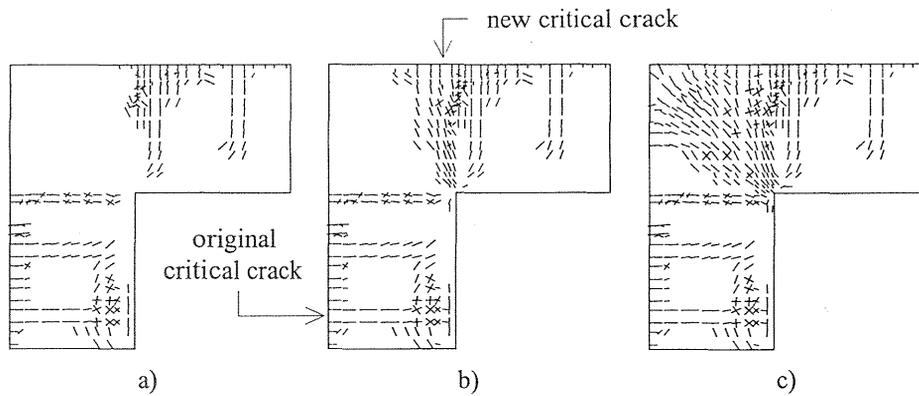


Fig. 7. Crack propagation in the corner area for the conventional reinforcement detailing at the load levels marked in Fig. 6.

of about 40 kN at which the reinforcement bars in the frame corner with new reinforcement detailing started to yield. Due to a greater amount of reinforcement in the sections adjacent to the corner (see Fig. 1), the load capacity of the frame corner with conventional reinforcement detailing continued to increase until yielding at a load level just below 50 kN was reached. The sudden loss of load capacity at a displacement of about 26 mm was caused by a redistribution of forces; the critical crack section moved from a region about 0.3 m below the construction joint (the section where the bent reinforcement bars, extending from the beam, ended) into the corner, see Fig. 7. This behaviour corresponded well with that observed for the frame corner with low amount of reinforcement and conventional reinforcement detailing.

Accordingly, the FE analyses showed that a frame corner with the conventional reinforcement detailing may have a somewhat higher load capacity than a frame corner with the new reinforcement detailing. However, this is only temporary since a redistribution of forces, reducing the load capacity to a level similar to that obtained when using the new reinforcement detailing, soon occurs. Thus, the load capacity is approximately the same, independent of the reinforcement detailing used.

In Fig. 6 the importance of the fracture energy is shown. It was found that the behaviour of the frame corner was approximately the same independent of the fracture energy used in the analyses. However, when using a lower value on the fracture energy a better correspondence in the load-displacement relation was obtained; thus, indicating that the fracture energy used in the analyses was somewhat too high. This can be due to several reasons such as different sizes and curing conditions of the specimens used in the tests and those used to measure the fracture energy.

The two-dimensional plane stress models used in the analyses worked well up to a load level corresponding to yielding of the reinforcement; the numerical problems encountered were mostly caused by the concrete in compression. It was found that once the ultimate compressive strength of the concrete was reached, the gradient of the descending softening branch was of great importance in further simulating the behaviour of the structure. This observation holds true also for the concrete in tension; a more gradual stress-strain relation describing the cracked concrete introduces fewer numerical problems in the analyses. Since the fracture energy and the tensile strength influence the gradient of the stress-strain relation, the value of the fracture energy can have considerable effect on the feasibility of simulating a problem. Accordingly, fewer numerical problems were encountered when the reinforcement bars were modelled using truss elements in combination with interface elements than when modelling the reinforcement bars using embedded reinforcement. This was because of the more gradual stress-strain relation obtained for the concrete softening when dividing the ultimate crack opening by the finite element length instead of the mean crack spacing.

4 Conclusions

The tests and the FE analyses conducted have shown that the conventional and the new reinforcement detailings for practical purposes are equivalent for a frame corner structure with a low amount of reinforcement. Comparisons of tests and FE analyses indicate that this is also the case for a frame corner with a high amount of reinforcement. Thus, the tests and analyses support the idea that the new alternative is suitable to use instead of the conventional reinforcement detailing.

The FE analyses have shown that the stiffness of the structure is affected by the bond-slip relation up to the point at which the steel reinforcement starts to yield: the higher the stiffness of the bond-slip relation, the higher the stiffness of the structure. When rupture of the reinforcement bars is the cause of final failure, a weak bond-slip relation has a positive effect on the deformation capacity of the structure. However, the maximum load capacity is relatively unaffected by the bond-slip relation.

The weakness of the construction joint affects the structural behaviour of the frame corner only in the initial cracking stage; its effect on the maximum load capacity is negligible. The structural behaviour of the frame corner was approximately the same independent of the fracture energy used in the analyses.

The BFGS secant method was found to be a comparatively robust iteration method, well suited for the kind of FE analyses carried out in

this study. Fewer numerical problems were encountered in the analyses when separate elements were used to model the reinforcement bars; this is due to the lower gradient in the stress-strain curve used for the cracked concrete.

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6 References

- CEB (1993) **CEB-FIP Model Code 1990, Design Code**. Thomas Telford, Lausanne, Switzerland, 437 p.
- Gylltoft, K. (1983) **Fracture Mechanics Models for Fatigue in Concrete Structures**. Division of Structural Engineering, Luleå University of Technology, Doctoral Thesis 1983:25D, 210 p.
- Johansson, M. (1996) **New Reinforcement Detailing in Concrete Frame Corners of Civil Defence Shelters: Non-linear Finite Element Analyses and Experiments**. Division of Concrete Structures, Chalmers University of Technology, Publication 96:1, Göteborg, 83 p.
- Plos, M. (1995) **Application of Fracture Mechanics to Concrete Bridges - Finite Element Analyses and Experiments**. Division of Concrete Structures, Chalmers University of Technology, Publication 95:3, Göteborg, 127 p.
- Swedish Rescue Services Agency (1994) **Shelter Regulations, SR** - English edition. Statens Räddningsverk, Publication B54-168/94, 94 p.
- TNO (1993) DIANA-5.1. TNO Building and Construction Research - Department of Computational Mechanics, Delft, The Netherlands.