Performance of an office building in fire

Nandor Mago, Charles Clifton, Martin Feeney and Tim Porter

NZ Heavy Engineering Research Association, Holmes Fire & Safety, Auckland, NZ

Abstract: *The fire resistance of steel framed multi story buildings and their ability to withstand exposure to fully developed fire conditions without the need for all structural members to be protected with insulating material (passive fire protection) has received significant attention in recent years. This has been based around determining patterns of partial passive fire protection in accordance with sound fire engineering procedures that will ensure satisfactory performance in severe fires. A suitable procedure is the Slab Panel Method (SPM), which has been developed from a programme of small and large scale experimental testing under both Standard Fire (ISO 834) and natural fire conditions, in conjunction with implicit and quasi-static explicit simulations. The Abaqus experience gained from this applied research is utilized to full extent in the current design study. In this paper the application of the SPM design approach is briefly outlined for a new 12 level office building under construction in Auckland's central business district, followed by details of the FEA undertaken to validate the SPM results. The outcome of the performance based design process is a steel structure with partial fire proofing. Structural elements which are critical for structural stability are protected with conventional fire proofing materials, and the floor beams for which fire proofing provides no improvement in structure performance are designed without this passive fire protection. Abaqus simulations are presented and benchmarked against the design.*

Keywords: *Buckling & Collapse, Design, Fire, Concrete*

1. Introduction

The design of composite steel-reinforced concrete slabs in multistory steel buildings for fire is either based on a prescriptive approach based on Standard Fire Test conditions and element performance or based on the application of performance based fire engineering design. The former is easy to apply, however does not accurately reflect the behavior of the overall steel structure in fire and does not allow the reserve of strength available from such systems to be utilized. The SPM (Clifton, 2006) is one of the emerging performance based methods developed over the last few years. It calculates the inelastic reserve strength available from a floor system responding in two way action. The method is based on the Cardington full-scale office building fire tests (Kirby, 1998), Colin Bailey's postulated tensile membrane action model (Bailey, 2000), and six slab panel floor systems that were experimentally tested in New Zealand (Lim, 2002). This was followed by extensive implicit and explicit quasi-static simulations (Mago, 2005).

In the current case study, the SPM was applied to determine the most cost effective treatment of the secondary cellular beams. The SPM predictions of peak deflection under fire were investigated by more accurate Abaqus/Explicit simulations for a range of design fire severities. The Britomart East building (36,000 square metre) is a twelve level building used predominantly for offices. It is located above an underground Train Station in Auckland's CBD, which has dictated the column layout and aspects of the lateral load resisting system. Although SPM is a key part of the fire engineering design, it is only briefly outlined in this paper. Instead, emphasis is given to the challenges of the structural analyses that have been performed on part of the building using the **`*CONCRETE DAMAGED PLASTICITY`** material model.

2. Application of the SPM

The Slab Panel Method is written for application to two-way elements of the floor system known as slab panels. Any floor support beams within this region are unprotected for fire. The edges of these panels are required to carry the loads from the slab panel when it deforms in fire, and the edge supports are required to undergo very limited deflection relative to that within the slab panel itself. To achieve this they are either protected or are sufficiently over designed for other purposes to carry the applied loads under design severe fire temperatures (over 800 ºC).

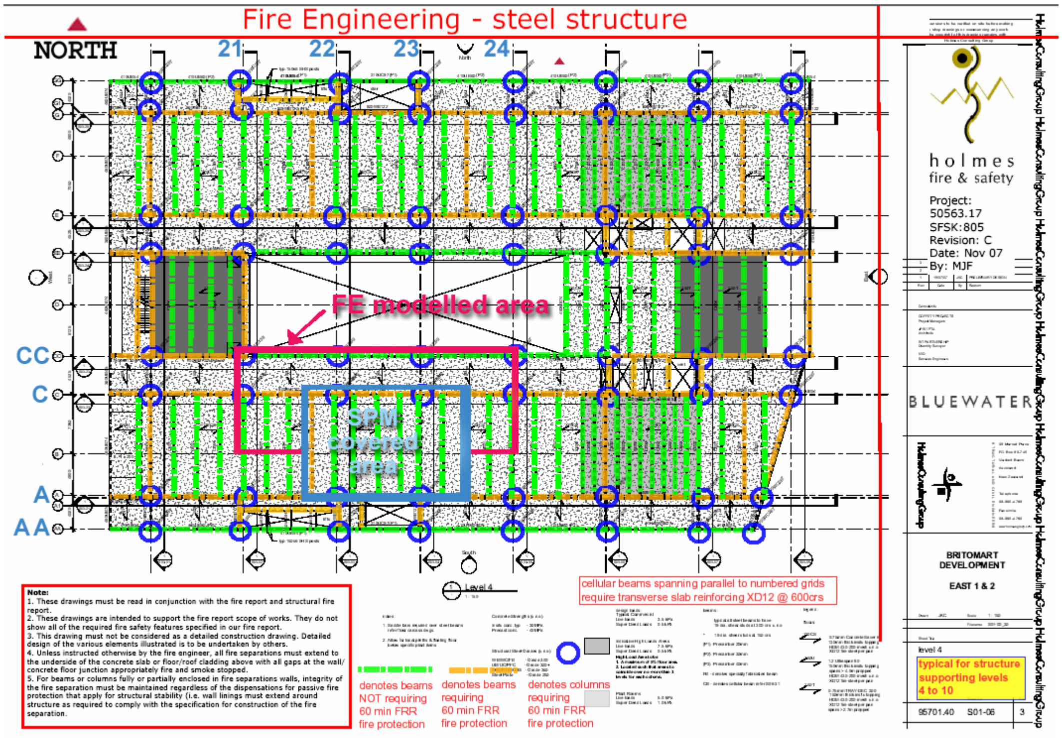

Figure 1 Typical floor plan shows the fire engineering design and the part of the building captured by FEA and SPM. Note, North is top of the plan.

Figure 1 shows a typical floor plan of the building. Along grids A and C there is a rigid moment resisting frame providing lateral restraint to the structure in the East-West direction. Slab panel supports in the East-West direction are the beams along grids AA, A, C and CC. The principal supports are along grids A and C and these beams do not quite have sufficient strength to support the slab panel without protection. The beams along grids AA and CC are shown to have sufficient strength to support the slab panel actions if unprotected, provided the connections remain rigid in fire, and this approach has been taken.

In the North-South direction between grids A and C, the initial design was based on secondary beams comprising 460UB74 sections, which were sized to meet serviceability stiffness requirements. To make the slab panel concept work, every 7th beam was designated as a slab panel edge support beam and suitably protected. When used in this way, these edge beams carry significant additional load from the slab panel action under fire emergency conditions. However, these beams were subsequently changed to cellular beams, which are customized beams formed from two tee sections cut and welded to form a deep, light beam with regularly spaced circular openings in the webs. The resulting beams were deeper and lighter than the UB sections - meaning that their reserve of strength under fire emergency conditions is much lower and was not adequate to support the full design load attributable from SPM action as required (Clifton, 2006). This required a re-evaluation of the SPM application, with the purpose of the FEA then being to show whether this application is valid. The revised application was on the following basis:

1. The slab panel support beams on all four sides can support the full fire emergency design load at their elevated temperatures.

2. The slab panel support beams in the North-South direction can support the lesser load associated with development of the yield-line mechanism associated with slab panel development at their elevated temperatures.

3. The slab panel support beams in the North-South direction will not undergo lateral buckling in severe fire conditions and are laterally braced to achieve this.

Figure 1 shows a typical slab panel analyzed on this basis. The slab panel support beams are protected to achieve a FRR of 60 minutes in accordance with NZS 3404 1997/2007 Section 11.

In terms of the design structural fire severity, a range of severities were determined and the SPM applied to the most severe of these. The average structural fire severity (equivalent length of time of ISO 834 exposure) was 45 minutes, the maximum was 75 minutes and an 80% value was just under 60 minutes.

The final SPM derived design solution between A and C grid lines is with the beams along the North and South sides of the slab panels protected and every 7th cellular beam along the East and West sides protected and stiffened. These beams are identified in Figure 1 and Figure 3. Of particular interest are the protected cellular beams, which can be seen to be effective as slab panel edge supports and have been designed with sufficient strength to develop the yield-line pattern of deformation, which is fundamental to the SPM behavior. SPM solutions were also obtained for the slab panels between grids AA and A, CC and C for which details are not presented herein. These solutions involved unprotected edge support beams on grids AA and CC.

The first objective of the FEA was not only to determine the adequacy of the above application of the SPM to the region of slab between grids A and C but also to determine the response of the smaller slab panels between grids AA and A and grids C and CC, respectively. For this reason the FEA captured the structural behavior between grid lines 21-24 and C-CC and half floor between C and A.

3. Finite element analysis and discussion

Figure 1 shows the modeled region. Since submodeling was not applicable in this case, the lateral support conditions were varied from free to restrained (symmetrical boundary conditions) along grid lines 21 and 24. In practice, all slab panels are laterally restrained to some extent, while the SPM assumes free boundary conditions in the plane of the slab. Symmetry was assumed in the mid plane between grid lines A and C.

The influence of boundary conditions is illustrated in Figure 2 for the slab node (N2, Figure 3). The design solution and the acceptable solution deflections (C/AS1: 2006) are quite different, but the engineering correlation is as follows: Neither the 100mm or the 800mm permanent deflection can be tolerated, thus the floor system would need to be rebuilt in both cases, while collapse is avoided in both cases and the floors function as effective fire separation. The final solution involves sprinkler protection to minimize the likelihood of fully developed fire and associated structural damage.

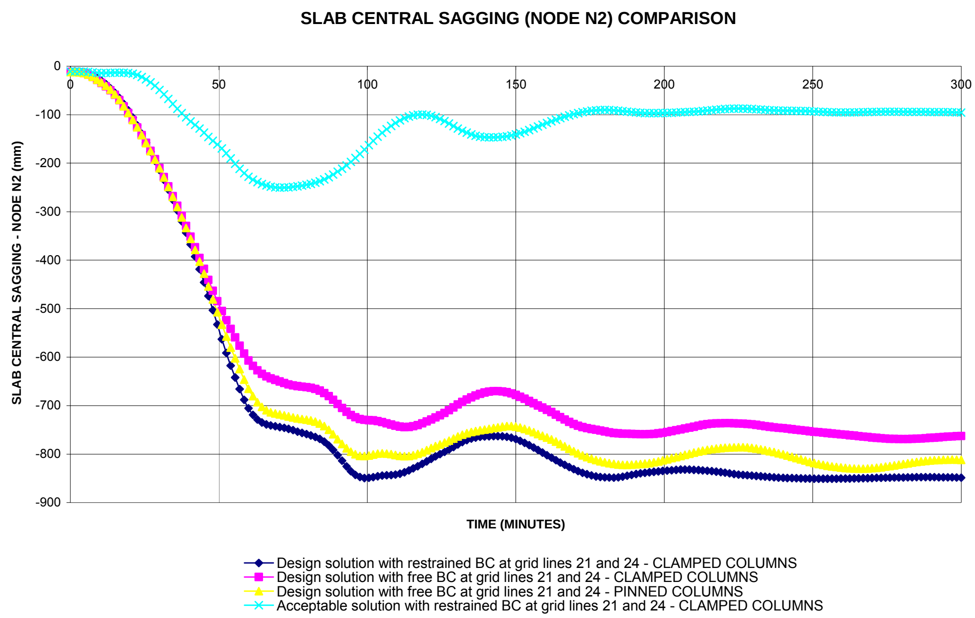

Figure 2 Influence of boundary conditions on slab central deflection on the design solution. The acceptable fire solution deflection is smaller, but both would require post-fire reinstatement of the floor.

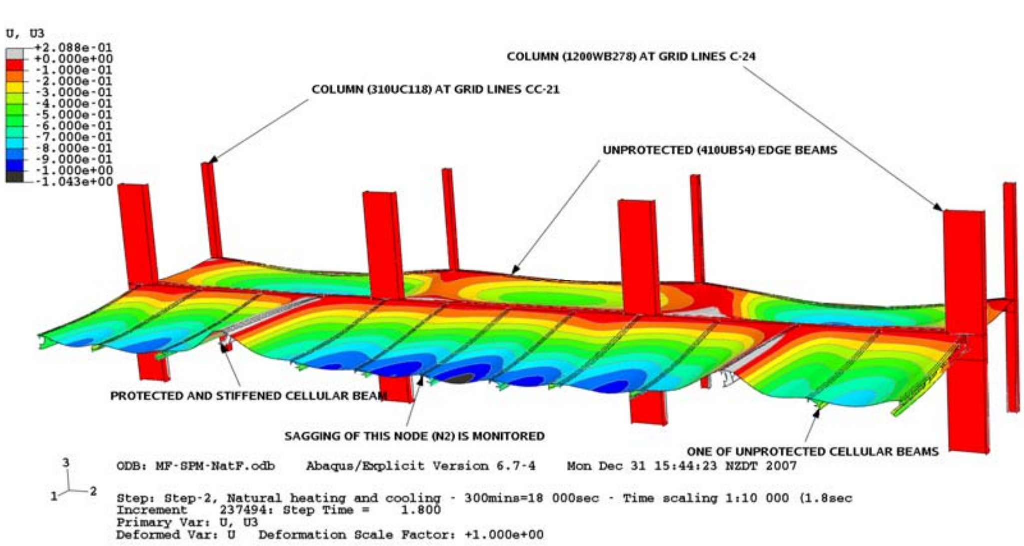

Figure 3 Design (SPM derived) solution. Actual deformed shape at the end of the cooling down period of the natural fire condition. Most of the cellular beams are unprotected.

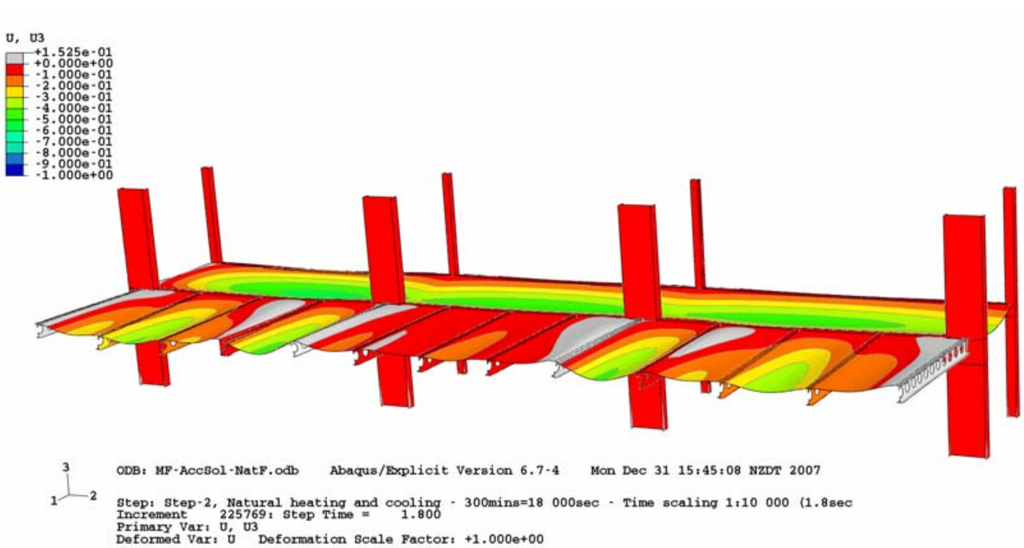

Figure 4 Acceptable fire engineering solution (C/AS1) with all steel members protected. Actual deformed shape showing the magnitude of vertical deflection at 300 minutes.

The floor system has been designed for a range of structural fire severities, up to the maximum structural fire severity of 75 minutes of ISO 834 Standard Fire exposure. These exposures were run only to the end of the heating curve and used time scaling of 1:1 000. In all instances the SPM shows a design solution for $t_{eq,max}$ = 75 minutes and the FE results confirm this.

A comparison of the predicted maximum slab panel deflections from the natural fire comparison shows that the predicted maximum slab central sagging from SPM is 1.17m compared with up to 1m from the various FE options. The most likely explanation of the difference is that the SPM does not take into account any stiffness against lateral (in plane) deformation of the slab panel edges, which is de facto present in the model by having continuous slab over the supporting beams.

The comparable design solution is that from applying the Acceptable Solution (C/AS1), for which all steel beams are protected. Figure 4 shows comparable deflected shape in the post-fire burnout condition.

Columns have been represented as extending to floor levels below and above the compartment in the FE model. At level four the columns are fixed or pinned as appropriate, while at level six they are axially loaded with design forces from the levels above. The boundary conditions allow the columns to extend only upwards. All beams are fully welded (`*TIE`) to the columns, while the cellular beams' web is connected to the primary/column web via bolts (`*TIE, NO ROTATION`). Equivalent reinforced concrete slab of 100mm thickness was used to represent the floor slab, which in practice is a composite steel-concrete trapezoidal profile. This approximation has been shown to be valid in the modeling of experimental testing undertaken as part of the SPM development, provided that the reinforcement position and area is adjusted to give equivalent load carrying capacity. The numerical analysis was simplified and adjusted to the available computing power by this approach.

The bottom of the slab is tied to the top face of all beams. Nine section points are specified through the composite shell thickness:

```
*Shell Section, elset=_PickedSet38, composite, temperature=3, offset=-0.5,
layup=Concrete 100mm in four layers
0.025, 3, Si-30MPa-STRAIN1, 0., Top layer
0.025, 3, Si-30MPa-STRAIN1, 0., Under top layer
0.025, 3, Si-30MPa-STRAIN1, 0., Above bottom layer
0.025, 3, Si-30MPa-STRAIN1, 0., Bottom layer
```

In addition to the self weight of the slab, it is loaded with 1.9KPa uniformly distributed vertical downwards payload, representing the superimposed dead load and fire emergency design live load. In the first step, the gravity loads, column forces and payload were applied in a smooth quasi-static explicit procedure, which time period was set to 1.1 seconds (being ten times the period of mode that it most resembles on the expected deflected shape). This step was followed by the fire loading step. Two cases were presented to the client. One of them is based on applying the prescriptive solution given in the Approved Document for Fire Safety (C/AS1) involving application of passive fire protection to all steel members. The second case, as designed, is a more cost effective solution based on selective fire protection and web stiffening at quarter points of the cellular beams (CBs) comprising slab panel supports in the North-South direction, while leaving unprotected the CBs within the slab panel region. The edge beams were also left unprotected.

As stated in section 2, one of the aims of the FEA was to determine the adequacy of the modified application of the SPM as part of the design solution. However another equally important aim was to determine the likely response of the structure to the design natural fire application, including the post-fire condition of the structure and in particular the floor. This second aim required simulation of the structural behavior for approximately 45 minutes heating up and for the much longer 255 minutes cooling down period. The simulation of this long lasting fire condition is challenging in explicit codes, therefore “time scaling” and mass scaling were used to obtain the solution within a reasonable time frame (up to one day) with the available hardware resources and license tokens (10 Abaqus tokens were available on a HP xw8400, two quad-core CPUS, 8 GB RAM, 15k rpm. This configuration allows jobs to run with 6 CPUS).

The difficulties in performing successful highly non-linear analyses of concrete structures with temperature dependent material properties are well known. Implicit codes struggle to provide a convergent solution. From the authors’ experience this had been demonstrated by analyzing in detail six full slab panels of 4.3m x 3.3m size (Mago, 2004), which have been experimentally tested under 180 minutes of ISO 834 standard fire conditions. The experimentally recorded slab panel central sagging versus time curves were used in all cases to assess the validity of various FEA results. Moreover, good correlation between Abaqus predicted rebar and strain gauges measured nominal strains have been obtained up to 300C (being the maximum temperature rating for the used strain gauges) in some tests.

The exercise allowed first hand experience to be gained on the limitations of Abaqus/Standard in this regard (although this applies mostly to version 6.3 – likely to be applicable to the current version also, since the description of `*CONCRETE DAMAGED PLASTICITY` remained unchanged). It is to be noted that visco-plastic regularization did not help to obtain a convergent solution.

As the finite element model size and complexity increased it was necessary to adopt an explicit approach to allow the models to progress beyond failure in some of their regions, so large deflections could be captured in many simulations.

The explicit FE approach partly used time scaling and partly mass-scaling. In other words, the most computational efficiency is obtained when the fire loading step is broken into several (sub) steps and the time period of each these (sub) step and within mass scaling, is adjusted so that the ratio of ALLKE/ALLIE is within the generally accepted maximum 5-10% range for the whole duration of the quasi-static simulation. Generally the early rapid heating part of fire requires longer time periods, but occasionally sudden buckling of structural elements can also be dynamic, thus requires a (sub) step with longer duration when it occurs.

In case of the Britomart office building fire simulation, time scaling to 1:10 000 was necessary to adopt. This means that the 300 minutes of elevated temperature condition was represented with time period of 1.8 seconds (while the stable time increment was in order of 3e-6 seconds).

```
*Step, name=Step-2
Natural heating and cooling - 300mins=18 000sec - Time scaling 1:10 000
(1.8sec explicit simulation time)
*Dynamic, Explicit
, 1.8
*Bulk Viscosity
0.06, 1.2
```

```
** Mass Scaling: Semi-Automatic
**               Whole Model
*Variable Mass Scaling, dt=4e-06, type=uniform, frequency=500
```

This uniform mass scaling resulted in a slightly dynamic solution at the start of the fire steps, with ALLKE/ALLIE being up to 30%, however this is short-lived and its influence has been shown to be negligible, but it can be corrected by partitioning the step, as described above. Such a large time scaling has not been used before by the author. Therefore it was desirable to assess its physical meaningfulness on other known solutions. The previously experimentally tested and extensively analyzed/validated Speedfloor FE model (Mago, 2005) was chosen for this purpose. The results are given in Figure 5.

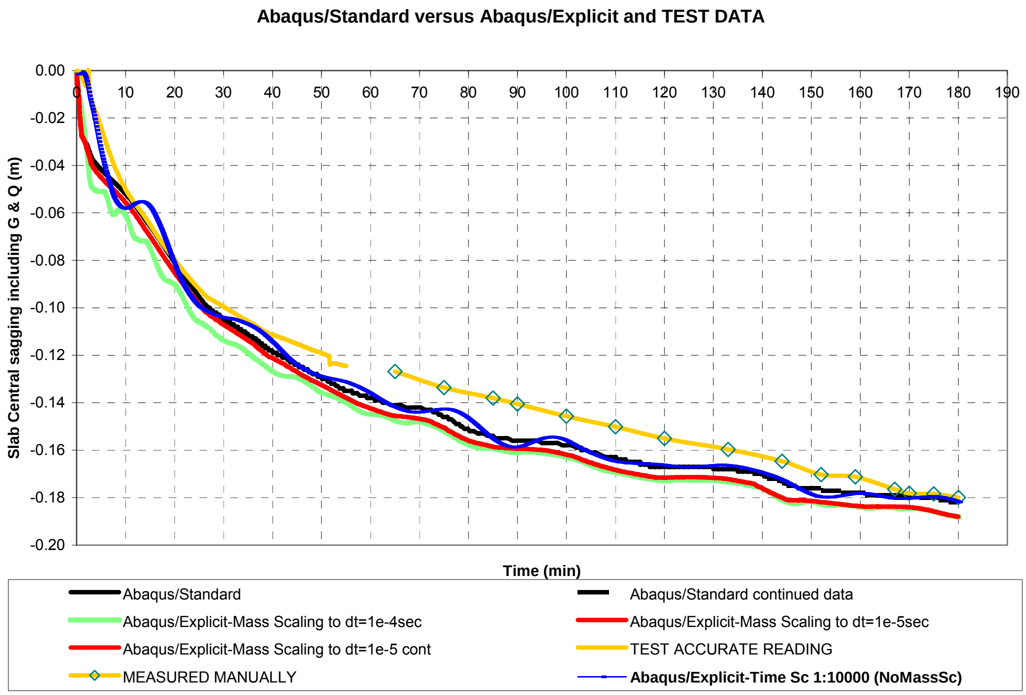

Figure 5 Comparison of slab central sagging from several FE options and test data of Speedfloor ISO 834 Standard Fire Test. SPM predicts 173mm at 180 minutes (Clifton, 2002).

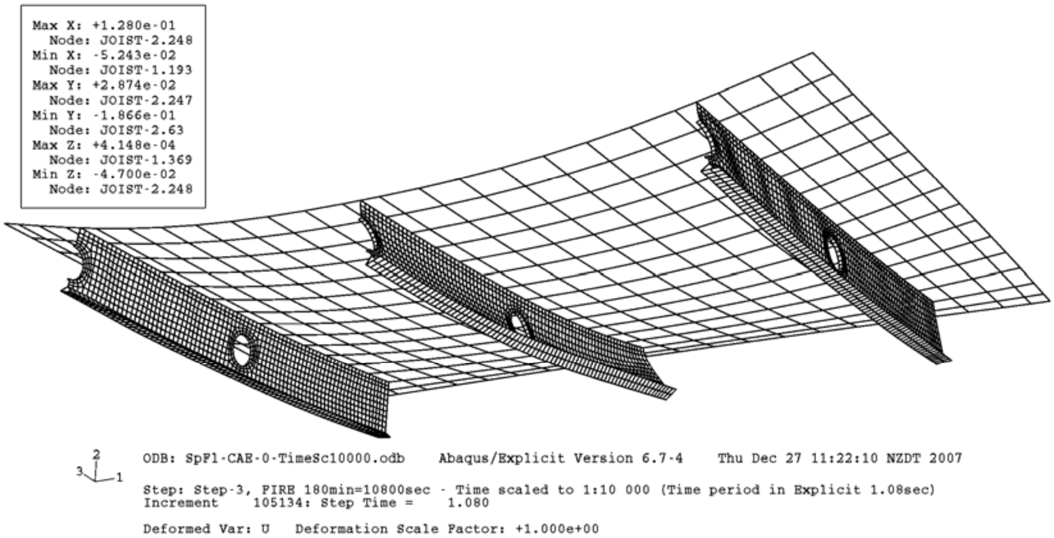

Figure 6 Actual deformed shape of half of the Speedfloor slab at 180min of Standard Fire loading.

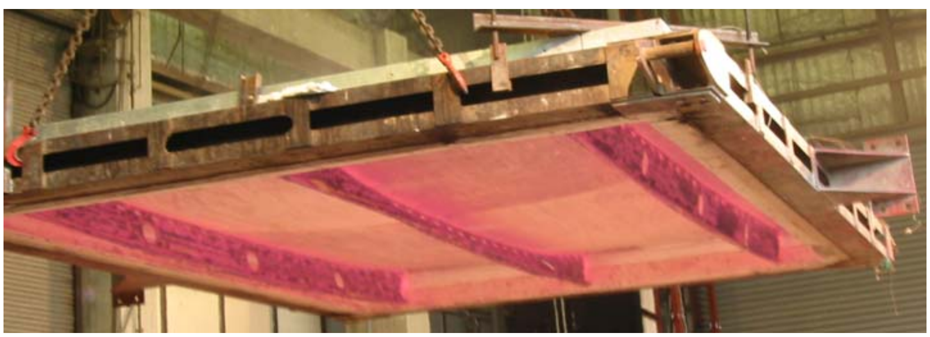

Figure 7 Speedfloor: Deformed shape after the fire test shows joists glowing and the buckled shape of the mid joist as it was lifted up some 10 minutes after the furnace was shut down.

The comparison of deformed shape and slab central sagging is very good. This gave us confidence in applying such a time scaling on the Britomart model. Guidelines in this context have not yet been found in the literature, but a threshold must exist for the time scaling approach beyond which the static equilibrium of internal and external forces fails.

As a final remark regarding the experimentally tested slabs it can be concluded that in many cases even the observed cracking pattern on the upper and lower face of the reinforced concrete slab could be identified/visualized in the form of Maximum Plastic Strain (PE).

The **`*CONCRETE DAMAGED PLASTICITY`** model is robust for these type of simulation, but the user must be aware of several pitfalls. Some of these issues are listed below:

- There is a mesh sensitivity issue present. Refining the mesh of the concrete parts does not lead to a unique solution and the tuning of tension stiffening parameters is required for each particular case. For significant regions of plain concrete the fracture energy approach should be used but this is not recommended when there is reinforcement. The implicit and explicit solvers abort with the reporting of excessive element distortions. Thus, in case of reinforced concrete slabs, the tensile stress versus tensile strain relationship is recommended with the "reasonable" estimation of concrete tensile strength, which is in the order of 5-10% for normal weight siliceous concrete, used in this case study.

- The temperature dependent concrete compressive stress versus inelastic strain curve has a major influence on the fire response of the structure. There is a considerable range for the strain at the maximum compressive stress, due to various ways of testing concrete specimens (ENV 1992-1-2: 1995), but this "scatter" has been set at the upper limit recently (ENV 1994-1-2: 2005(E)). Changes in this input stress-strain curve affect the predicted deflections versus time outputs, even for small problems.

- Despite extensive search, the authors have not found details for the less common input data as the concrete dilation angle in the p-q plane for various strengths of concrete and for elevated temperatures, thus estimates have been made. Neither general guidelines regarding the numerical values of **`*CONCRETE COMPRESSION DAMAGE`** and **`*CONCRETE COMPRESSION DAMAGE`** have been obtained.

The influence of some of these issues on the vertical displacement of node (N2, Figure 3) is given in Figure 8. This comparison is interesting from the numerical point of view.

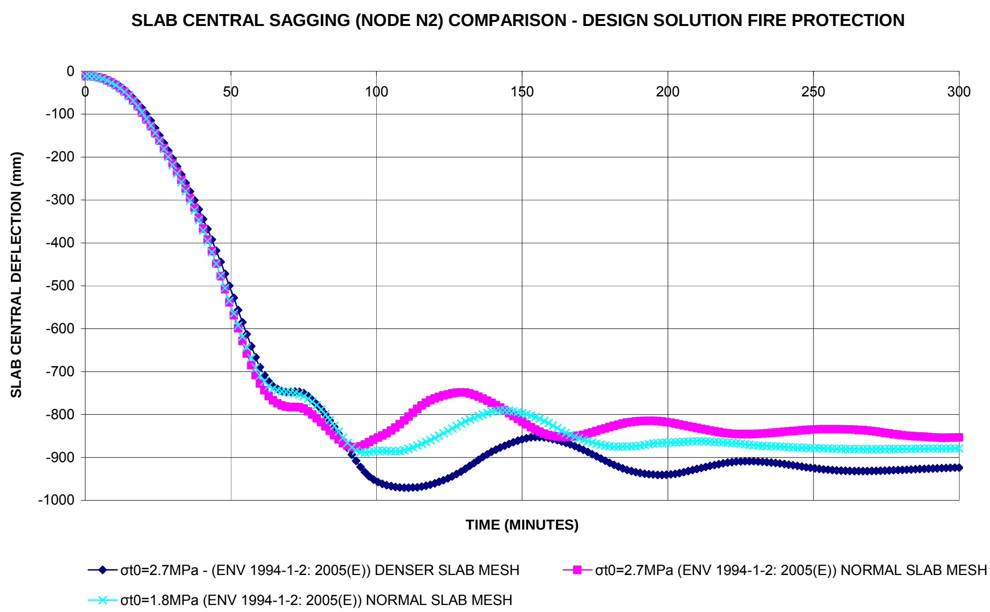

Figure 8 Influence of concrete slab mesh density and estimated "tension stiffening".

On the other hand correlation with the predictions of SPM should be seen in the context of understanding the applied boundary conditions on the model and the assumed free slab boundary conditions in the SPM method, as previously mentioned.

4. Conclusion

The Slab Panel Method is a fire engineering design procedure which allows the inelastic reserve of strength available in severe fire from a composite floor system supported on steel beams to be determined and used in design. When the original design solution for the building presented in this paper was envisaged, it was anticipated that the FEA would not be a critical part of the design solution but instead would offer a means of comparing the accuracy of the SPM solution with a more accurate Abaqus/Explicit simulations.

However the decision to replace the hot rolled secondary floor beams (i.e. those in the North-South direction) with cellular beams forced a re-evaluation of the application of the SPM and the role of the FEA. With this change, the secondary slab panel support beams no longer have the strength to carry the fire emergency loading as required by the current SPM procedure (Clifton, 2006). This has required a significant modification to the application of the SPM and the FEA has provided the means of assessing the adequacy of this modification.

Time scaling of up to 1: 10,000 is proved to be a valuable approach in speeding up the explicit quasi-static simulation. It has shown that the modification is appropriate and that the modified SPM still provides a conservative determination of the fire resistance of the floor system. Therefore, the Abaqus results are an integral part of the design solution for this project.

The results show the following:

1. Both the SPM solution and the acceptable solution provide a structural system that meets fire safety requirements of the New Zealand Building Code.

2. Both solutions - if subjected to fully developed average fire conditions -, will end up with excessive post-fire deformation requiring repair or reinstatement. While the maximum during fire and post fire deformations are larger with the SPM solution, in both instances they are sufficient to require repair or reinstatement of the floor system.

3. Abaqus/Explicit has shown to be a valuable tool to underpin the validity of the deflection predictions of the more conservative Slab Panel Method, generally used by fire engineers.

5. References

1. Bailey, C.G., “Design of Steel Structures With Composite Slabs at the Fire Limit State,” Report 81415, UK Building Research Establishment, Watford, England, 2000.
2. Clifton, G.C., “Steel Design Construction Bulletin No. 70,” New Zealand Heavy Engineering Research Association, 2002.
3. Clifton, G.C., “Design of Composite Steel Floor Systems for Severe Fires,” HERA Report R4-131: 2006, New Zealand Heavy Engineering Research Association, 2006.
4. “C/AS1: 2001 and Amendments 4 & 5,” Acceptable Solutions for Fire Safety Clauses, C1-C4, 2005.
5. Kirby, B.R., “The Behaviour of a Multi-Story Steel Framed Building Subject to Fire Attack - Experimental Data,” British Steel Swinden Technology Centre, UK, 1998.
6. Lim, L., and C. Wade, “Experimental Fire Tests of Two-Way Concrete Slabs,” Fire Engineering Research Report 02/12, University of Canterbury School of Engineering, Christchurch, New Zealand, 2002.
7. Mago, N., and G.C. Clifton, “Stage 2 Development of the Slab Panel Design Procedure,” HERA Report R4-118, New Zealand Heavy Engineering Research Association, 2004.
8. Mago, N., and G.C. Clifton, “Realistic Simulation of Reinforced Concrete Slabs Under severe Fire Attack,” Proceedings of NAFEMS World Congress 2005, Glasgow, UK, 2005.
9. NZS 3404: 1997 “Steel Structures Standard, Amendment 1: 2001 and Amendment 2: 2007,” Standards New Zealand, Wellington.