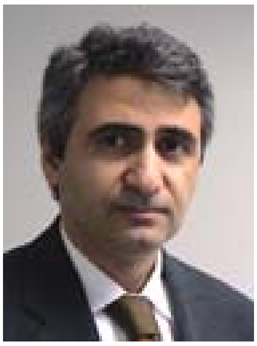
NON-LINEAR STRUCTURAL INTEGRITY ANALYSIS



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Dr. Ahmad Rahimian., PE, SE is President of Cantor Seinuk Structural Engineers in New York City. An expert in the behaviour of steel and concrete structures under seismic and wind loading, he has written numerous papers, and lectured widely in various professional societies and universities on design of tall buildings. He has been extensively involved in the design and engineering of stadiums and buildings worldwide. Recently, he directed the structural engineering of the Trump World Tower, the tallest residential building in the world and the Torre Mayor in Mexico City, the tallest building in Latin America. He holds a US patent in seismic protective design. Dr. Rahimian currently serves as an Adjunct Professor at The Cooper Union, School of Architecture.

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ABSTRACT: The provision of British Standard BS5950 is discussed. The intent of the current standard on structural integrity provision is to localize the damage as a result of removal of one member (i.e.: column). A new method for enhancement of the structural integrity requirement is presented here. This method allows localizing the damage in an event of multiple column removal and therefore eliminating the likelihood of disproportionate or progressive collapse. The method utilizes the interaction between beam and slab elements in a three dimensional space by considering the membrane forces generated into the diaphragm by the geometric action of the deformed structure. A series of three dimensional non-linear finite element analyses were performed to simulate the behaviour of the floor system in absence of supporting columns.

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INTRODUCTION

The aim of the disproportionate collapse criteria of the UK Building Regulations and material codes of practice is to ensure that buildings are generally robust and that a local incident does not cause large-scale collapse.

This paper presents a method for enhancing the structural integrity of high rise buildings beyond current practice. This approach focuses on redundancy and enhancement of the alternate load paths. This approach requires three dimensional analysis of the floor framing considering geometric and material nonlinearity.

This approach was developed for enhancing the structural integrity of a high-rise building in the United Kingdom beyond the British Standard. The criterion was set to be that the overall integrity of the structure should not depend on the integrity of any one or two columns.

The design process was developed using three dimensional finite element nonlinear large deformation pushover analyses for various column removal scenarios.

CURRENT REQUIREMENTS OF BRITISH STANDARD

Current British Standard (1) has a descriptive integrity requirement which implies that the descriptive requirement, if met, can accommodate the removal of any one column without initiating a progressive collapse. The descriptive provisions of British code of practice include requirements for internal ties, peripheral (edge) ties and vertical (column) ties with specified capacities. The descriptive criteria, while adequate, will not stand the scrutiny of a conventional analysis which is limited in its prediction due to inherent simplified assumptions on structural and material behaviours. However, recent tests carried out at the University of Berkeley, California, demonstrated that such structures can perform successfully (2).

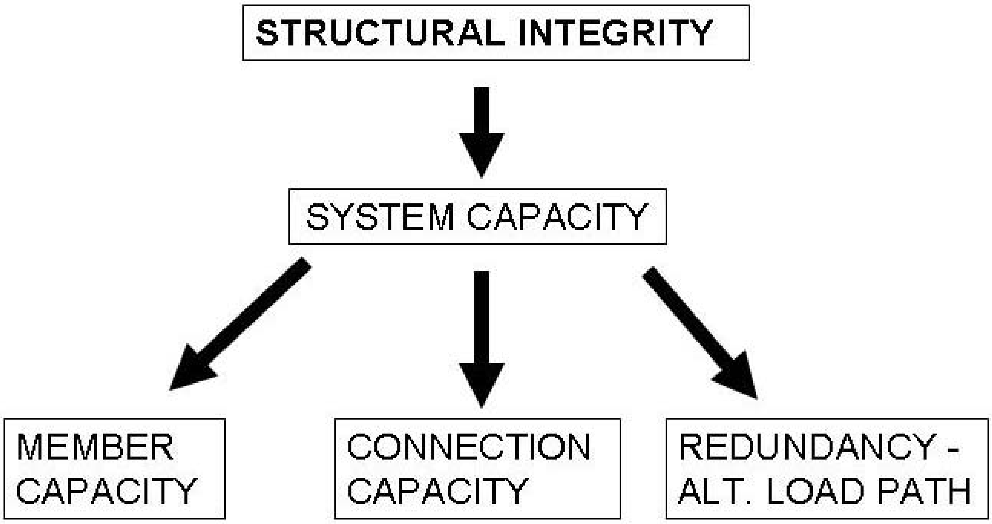
The few standards that address the issue have in general descriptive provisions instead of performance provisions. The descriptive provisions try to enhance the structural integrity without addressing any specific threat or structural performance. The performance provisions, while computationally more demanding, have the advantage of addressing directly the structure's behaviour under a given scenario.

ENHANCED STRUCTURAL INTEGRITY CRITERIA

For this specific project, the current British structural integrity criteria were enhanced to tolerate removal of any two columns within the building.

Generally, any measures with respect to structural integrity aim to enhance the system capacity. The system capacity enhancement is achieved either by enhancing member capacities, ductility or introducing alternate load paths or redundancy.

In this specific project, the goal was to enhance the structure redundancy by expanding on the alternate load path capacity. In order to ensure that the alternate load paths have adequate capacity to transfer the loads, member strength and ductility requirements were reviewed and upgraded.



The two column removal in essence is similar to the single column removal except imposing a higher demand on the remaining structure. In order to activate the secondary load paths the structure will go under a large deformation to the extent that is necessary to engage the self-equilibrating catenary behaviour of the floor system. This obviously requires that all other modes of failures have a higher load carrying capacity.

Figures 1 and 2 show the two-dimensional catenary concepts for single and double column removals. In two-dimensional analysis tie forces at the end of the catenary are required to achieve equilibrium. In general, the horizontal component of the tie force cannot be handled by any reasonable sized columns.

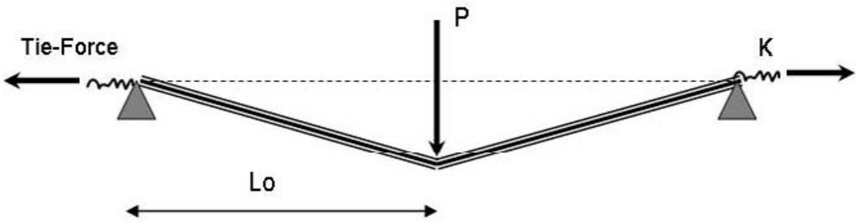


Figure-1: Catenary equilibrium for single column removal

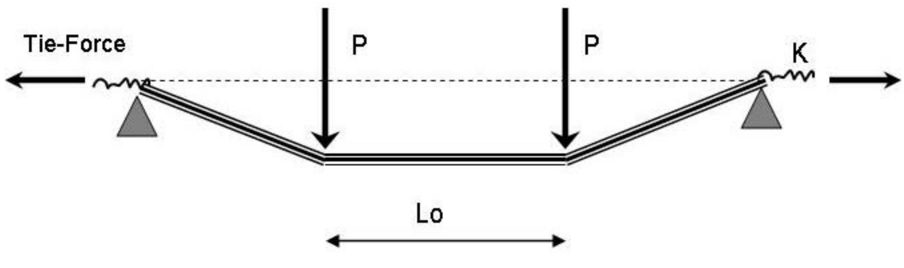


Figure-2: Catenary equilibrium for double column removal

Figure 3 shows the relationship between the two secondary load paths, i.e. beam action and catenary action as a function of beam rotation. The M/MP line shows the beam flexural action and the T/Ty shows the catenary actions. Initially the system under low level of loads acts as pure bending element and as the load and the rotation increases the system will convert itself from a bending element to a catenary element.

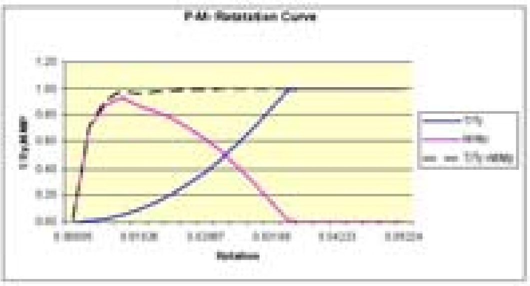


Figure-3: Interaction between "Beam Action" and" Catenary Action"

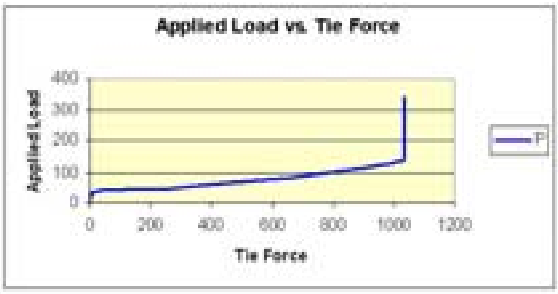


Figure-4: Relationship between applied load and catenary tie force

Figure 4 shows the relationship between vertical load and horizontal tie force for single column removal scenario.

In a two dimensional catenary system the system integrity depends on the capacity of the support to resist the horizontal component of the force. Generally, columns are not designed to receive lateral loads of the magnitude required for the equilibrium of a catenary system.

The advantage of a three dimensional behaviour is that the equilibrating forces are internal to the system. In principle the floor system goes through a deformation that reshapes the floor plate into an inverted dome or a dish, see Figure-5. As a result, the equilibrium of the system does not depend on the capacity of the catenary tie forces at the support. In other words, the radial tensile forces created as a result of the dishing action is balanced with compressive hoop stresses of the composite floor system.

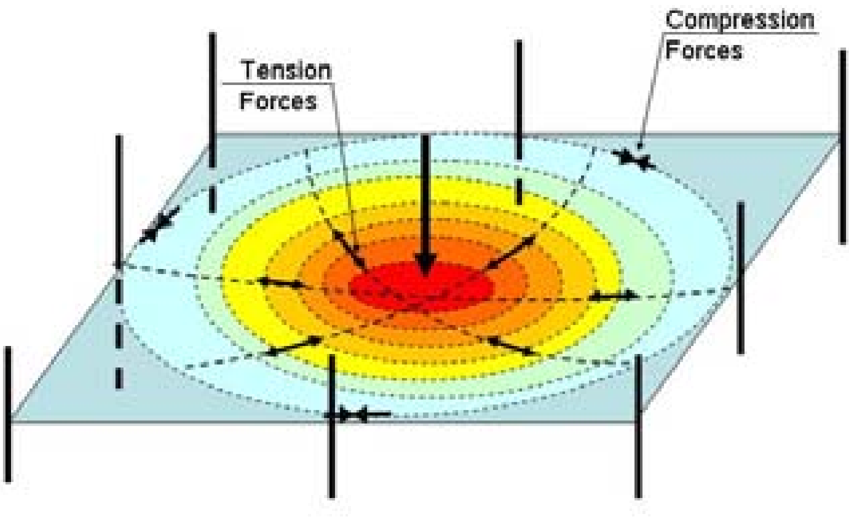


Figure-5: Three dimensional catenary shell action of the composite floor a system.

CASE STUDY

The building is 35-stories 165 meters tall. The building lateral system is comprised of three independent concrete core wall systems surrounding the stairs, elevators and mechanical zones. The floor framing system is steel construction with composite metal deck and concrete slab supported by steel columns and concrete walls, see fig.6. Average column spacing is 9 meters.

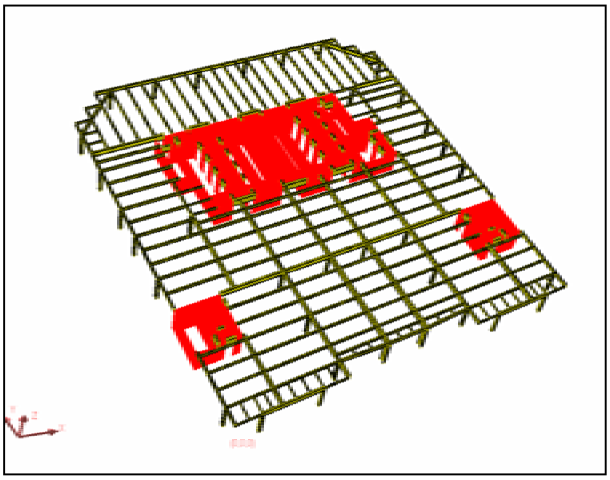


Figure-6: typical floor framing

GEOMETRIC AND MATERIAL NON-LINEAR FINITE ELEMENT ANALYSIS

To study the behaviour of the structure, a three-dimensional model was created using the "LARSA Integrated Linear and Non-linear Finite Element Analysis and Design" computer program. The beam element includes the effects of both geometric and material non-linearity. The analysis involves continual iterations of the stiffness matrix at points along the length of the beam as the load is gradually applied. This iteration process continues after the beam has yielded and redistributes the stresses to the adjacent structure as the load is increased. This analysis involves developing appropriate demand and capacity curves that are utilised to assess - and ultimately prevent - structural collapse.

A three-dimensional model of a whole typical structural floor was created as shown in Figure-6. This structural model was then analysed for a series of column removal scenarios. It was considered that when a column was removed the line of columns above would act as a hanger so that every floor structure would be forced to behave in a similar fashion to the floor being analysed; the hanger columns would effectively become redundant.

The concrete floor slab was modelled as thin shell elements connected to the steel floor beams. The tensile membrane stress in the concrete slab was monitored, and cracking of the concrete slab was considered. The cracking of the concrete in the principal tensile direction reduces the stiffness of the shell in the radial direction. This mechanism, due to strain compatibility requirement, sheds the tensile radial forces to slab reinforcement as well as the steel framing grid acting in the radial direction.

LIMITING CRITERIA

It was necessary to establish criteria in order to confirm the structural integrity after the columns are removed. The first criterion was to ensure that the columns adjoining the removed columns were not overloaded. This is achieved by performing strength check on the remaining structure, especially the adjacent columns using a realistic extreme event service load.

Alternatively this can be carried out by checking manually that columns adjacent to the removed columns can support the new load due to enlarged tributary area.

In extreme events, maximum deflection is not directly a limiting criterion. The behaviour can be considered acceptable if the strain in the yielded beam is limited to prevent collapse. Consequently, a maximum strain limit of 5% was adopted rather than imposing maximum deflection criteria.

The distribution of forces to the adjoining structure was also monitored. The results show that the bending moment diagram, together with the catenary axial force, reflects the element yield diagram by showing that the–bending–moment beyond the beam's plastic capacity as being yielded. The beam axial forces also display the effect of catenary action by activating the surrounding grillage of beams and the concrete slab.

The floor plate stresses were closely scrutinised to prevent failure. The forces were distributed between the catenary action of the steel structure and membrane action of the slab. In the extreme event the concrete floor will act as a thin shell developing radial and hoop stresses. Concrete floor on metal deck can easily accommodate the compressive hoop stresses and its reinforcement was primarily upgraded to carry the tensile stresses. The tensile capacity of the concrete as well as metal deck is ignored.

LOADING CASE AND SCENARIOS

The pushover nonlinear analysis was performed under full dead load of the structure and superimposed dead load. 50% of live load was considered.

Using the above model, various column removal scenarios were analysed. Obviously, in each scenario a different load path is activated.

ANALYSIS RESULTS

While in each scenario a different load path is activated, the performances are similar and can be categorized in three groups i.e.: Interior column removal, Perimeter column removal, Corner column removal.

Interior Column Removal:

Figures-7 and 8 show the plate principal compressive and tensile stresses. Figure-9 shows plastic hinge formation in the main catenary framing member. Figure-10 shows the catenary forces in various framing members.

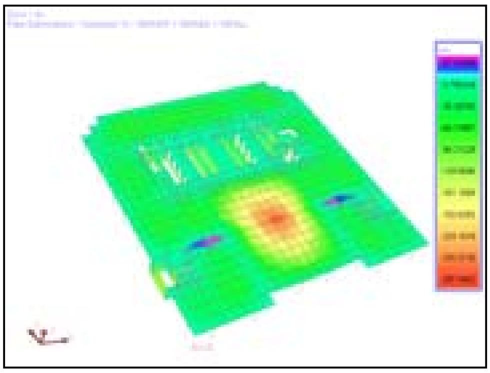


Figure-7: Principal compressive hoop stresses in the concrete floor plate

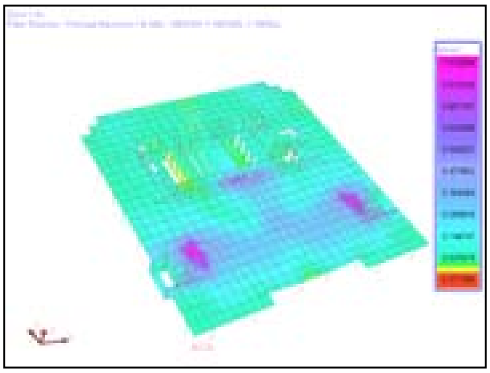


Figure-8: Principal tensile radial stresses in the concrete floor plate

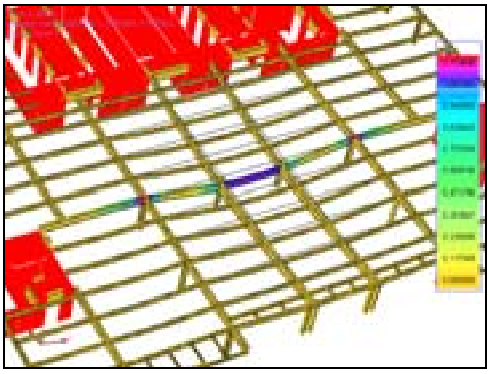


Figure-9: Plastic hinge formation in the main catenary framing element

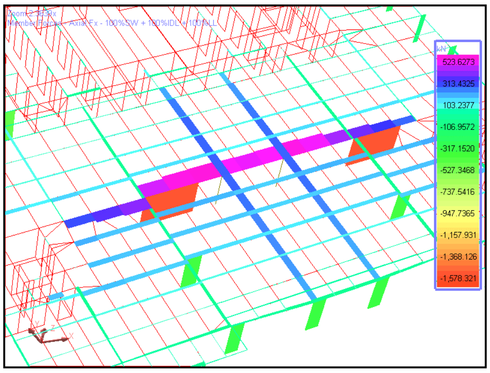


Figure-10: Catenary force diagram

Exterior Column Removal:

Figures- 11 & 12 show the plate principal compressive and tensile stresses. Obviously the exterior condition does not allow a complete formation of catenary shell action. As a result additional stress is imposed on the spandrel beams. Figure-13 shows plastic hinge formation in the main catenary framing member. Figure-14 shows the catenary forces in various framing members.

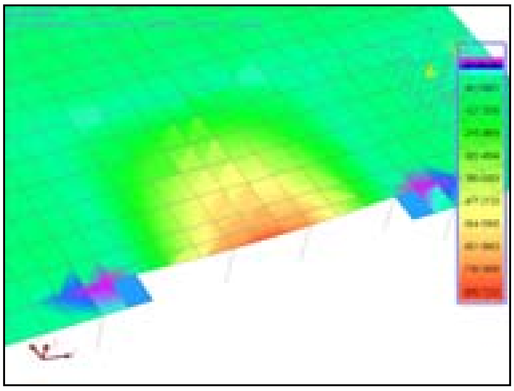


Figure-11: Principal compressive hoop stresses in the concrete floor plate

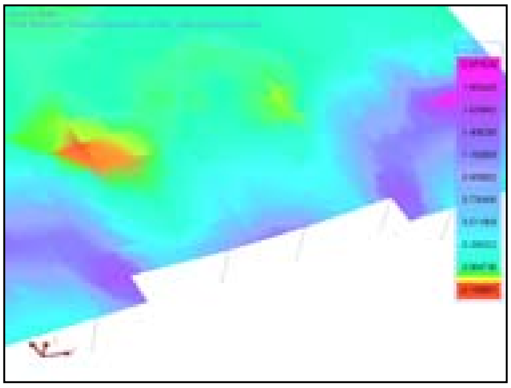


Figure-12: Principal tensile radial stresses in the concrete floor plate

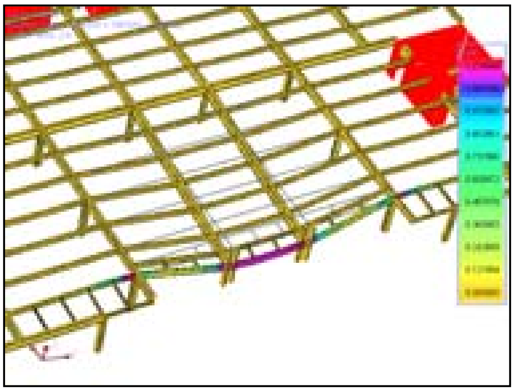


Figure-13: Plastic hinge formation in the main catenary framing element

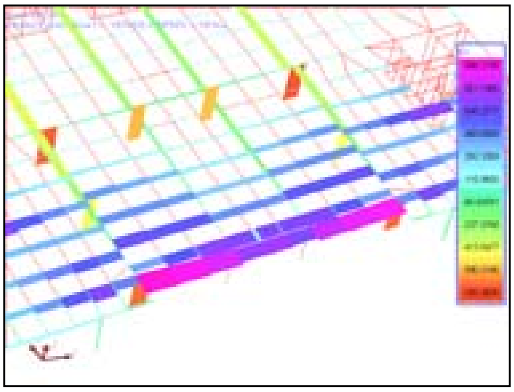


Figure-14: Catenary force diagram

The result of the analysis provides information for design and detailing as follows:

Beams & Girders:

Except in a few locations where beam sizes upgraded to meet the member force demand, majority of the floor framing sections were adequate.

Connection:

The analyses show that the beams functioning as catenary members need to have full capacity connection throughout their entire span. Therefore, connections of perimeter beams and interior girders were required to be upgraded to full plastic capacity. The secondary beam connections remained as simple shear connections, however, their capacities were checked to insure adequate transfer of the catenary axial forces.

Slab:

Compressive strength of concrete slab was adequate for compressive hoop stress demand. The tensile capacity of the slab system was enhanced by upgrading the mesh reinforcement and additional rebars at only critical locations.

Steel member strain:

Member strain was limited to 5%.

Floor Deflection:

Depending upon the location and the column removal scenario, the floor deflection varied from 250mm to 900mm.

The combination of the concrete cores, the floors connected to them and the continuously-designed perimeter frames creates a robust three-dimensional system that allows the structure to redistribute the forces in all directions, preventing a progressive collapse scenario resulting from the loss of columns. Of course, in this extreme event the structure will go into the plastic range with large deformations creating the required membrane and catenary forces to stabilise the system.

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4- Hysteretic Models for Cyclic Behaviour of Deteriorating Inelastic Structures, M.V.Sivaselvan, A.M.Reinhorn Multidisciplinary Center for Earthquake Engineering Research, November 5, 1999.