OpenSees Days Portugal 2014 - Abstracts

Workshop on Multi-Hazard Analysis of Structures using OpenSees

July 3-4

Faculty of Engineering of the University of Porto

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Welcome Message

Dear Participant,

The OpenSees Days Portugal 2014 Organizing Committee would like to welcome you to Portugal, to Porto and to the Faculty of Engineering of the University of Porto (FEUP). The OpenSees Days workshop is being held in Portugal for the first time and follows successful workshops held previously in Italy (2011), China (2012), and U.K. (2014). In the U.S.A. this workshop is organized every year. We are honoured to host this workshop at the campus of the FEUP and to welcome Dr. Frank McKenna, OpenSees founder, and the remaining lecturers, as well as some of the more active European developers.

The OpenSees Days Portugal aims to gather new users and assemble the OpenSees community in Europe to discuss some of the new features. The first day, entitled Getting Started With OpenSees is intended to give a major overview of the main features of the software, starting from a basic introduction to the structure of the program, introducing some basic tools and interpreter related issues. This is followed by a basic introduction to nonlinear analysis, with simple illustrations being given.

Several earthquake induced multi-hazard analyses using OpenSees are then addressed, such as blast analysis, fire analysis, and seismic analysis of structures. The use of parallel processing, HTC and HPC, which are of major importance due to the complexity of most of the systems required to perform multi-hazard analysis, are also presented. The first day ends with a video conference on the OpenSees Geotechnical features, delivered by one of its main developers, Prof. Pedro Arduino from the University of Washington, U.S.A.

The second day is essentially composed by the presentation of multiple case study examples. The first lecture, delivered by Dr. Frank McKenna, focus on adding code to the OpenSees framework, while the following lectures will be given by the participants of the workshop. The main goal of these presentations is to share recent developments, outcomes, challenges and to raise a fruitful discussion between the participants. In addition to the lectures given by Dr. Frank McKenna, Dr. André Barbosa, Prof. Asif Usmani, Dr. Lauren Stewart, and Prof. Pedro Arduino, a total of 12 presentations will be made at the workshop by authors from 7 different countries.

The remaining participants, approximately 40, come from 10 different countries. We are pleased to host such an inter-cultural workshop and hope that this serves as opportunity to launch new partnerships.

We would like to thank all the funding institutions that have made this workshop possible, namely the Portuguese National Science Foundation *Fundação para a Ciência e Tecnologia*, *FCT* and the University of Porto. We would also like to thank all lecturers, participant authors, general participants, and students for their contribution to the workshop. A special word of thanks goes to all who actively collaborated in the organization of this event at FEUP and to the secretariat of “Instituto da Construção” for their considerable help.

We sincerely hope you have a stimulating and fruitful workshop and wish you a very pleasant stay in Porto.

The Organizing Committee

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Table of contents

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Modeling Structures Subjected to Blast Loads

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SUMMARY

The incorporation of blast loading into analysis software is greatly needed for multi-hazard design and analysis. Currently there does not exist a framework whereby all hazard loads can be applied, the system analysed and then checked for response criteria. The lack of blast loads in software is due to a variety of reasons. First, there is a lack of knowledge and experimental data available to validate all of the structural behaviour needed to describe the response of the system. Secondly, the switch of timescales from very small (blast) to those normally used to check collapse is problematic from a convergence standpoint. This research aim is to begin to incorporate blast into OpenSees, with the ultimate goal to provide an opportunity to model for blast and other hazards in one cohesive framework. This paper presents comparisons to the most common blast analysis methods and it is hoped that incremental progressions will take place as more users become involved.

Key Words: *blast loading, SDOF, advanced analysis*

1 INTRODUCTION

“An *advanced* analysis method is one in which the nonlinear geometric and material effects are accounted for in the analysis of the structure as a whole in determining it's ultimate load carrying capacity. In addition, effects of local as well as overall global instability are considered such that it is not necessary to evaluate individual members subsequent to the completion of the advanced analysis [1].” Simply, all appropriate limit state design code requirements are incorporated into the analysis. Currently, tools necessary to carry out *advanced* analysis for blast are generally not available.

Typically, two methods are used for analyzing structures subjected to blast loads: detailed finite element models and simplified SDOF models. Detailed finite element models, created from commercially available software, are often effective in analyzing both global (i.e. deformations, curvature) and local (i.e. fracture, local buckling, etc.) responses of the structure. These methods, while accurate are generally computationally expensive especially when analyzing the system as a whole and limit states are not incorporated into the analysis. Simplified SDOF models can rapidly analyze structural components and be compared to limit states, but localized responses are often ignored in the analysis. Also, the option to analyze the transfer of forces between each member in a cohesive framework is none existent. This research aims to create the initial framework for an advanced analysis method for blast whereby structures could be analyzed as a whole and compared to representative design codes, such as those available for seismic and other loads.

2 BLAST LOADS & ANALYSIS

A typical blast environment is shown in Figure 1 and includes a spherical charge of weight of TNT, *W*, at a standoff distance, *R*, away from a structure of interest. Various types of high explosives (i.e. C4, ANFO, etc.) can be converted to an equivalent weight of TNT using a ratio of specific energies as shown in Eq. (1), where Q_{EXP} is the specific energy of the explosive of interest, Q_{TNT} is the specific energy of TNT and W_{EXP} is the weight of the explosive.

$$W_{TNT} = \frac{Q_{EXP}}{Q_{TNT}} W_{EXP} \tag{1}$$

When the explosive is detonated, a shock wave is formed and it moves outward from the charge towards the structure [2]. At the time the wave hits the structure, it undergoes reflection when the forward moving air molecules in the blast wave are brought to rest on the structure. These molecules are then further compressed by the moving molecules behind, inducing a reflected overpressure on the structure. An example of a typical reflected pressure-time history, $p(t)$, at a point on the structure is also shown in Figure 1, where p_r is the peak reflected overpressure. Values for peak reflected overpressure can be found in many ways including using equations derived by Rankine and Hugoniot and using graphical methods in UFC 3-340-02 [3].

The shape of the pressure-time history curve is characterized by a steep increase in pressure followed by an exponential decay, the shape of which is often determined by the Friedlander equation. The time of arrival, t_a, is the time at which the shock front arrives at the structure. The duration of the positive phase, T_p, is the time in which the pressure is above ambient. Similarly, the duration of the negative phase, T_n, is the time in which the pressure is below ambient. These durations are on the order of milliseconds and are typically much much less than the period of the structure of interest. In addition to peak reflected pressure, specific reflected impulse, i_r, is also an important blast wave parameter. Specific impulse, which from herein will be referred to as impulse, is the area under the pressure-time curve from arrival time to the end of the positive phase.

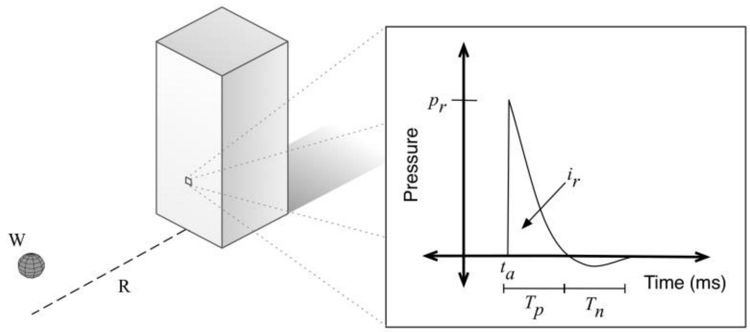


Figure 1: Typical Blast Environment and Pressure-Time History [4]

The very simplest analysis method for blast consists of reducing the structure to an SDOF model, which is a function of the resistance of the structure to the blast load as shown in Equation 2. For a simply supported member with no axial load, the beam is loaded with the blast pressure until a hinge forms in the midspan (at location with maximum stress). The resistance of the beam to the blast load can be, in the simplest cases, modelled as elastic, perfectly plastic where the transition from elastic to plastic behaviour is the resistance to the formation of the plastic hinge. The load mass factors, $K_{LM,}$ in the equation are functions of the boundary conditions and the loadings applied. Some most common combinations can be found in [5].

$$K_{LM} m\ddot{x} + R(x) = p(t) \tag{2}$$

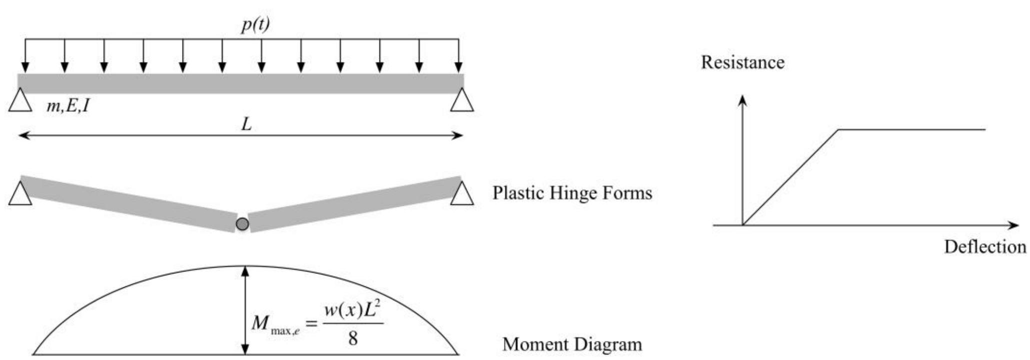


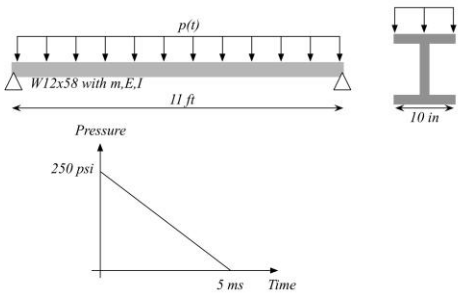
Figure 2: Blast Loading Behavior with Simple Resistance Function

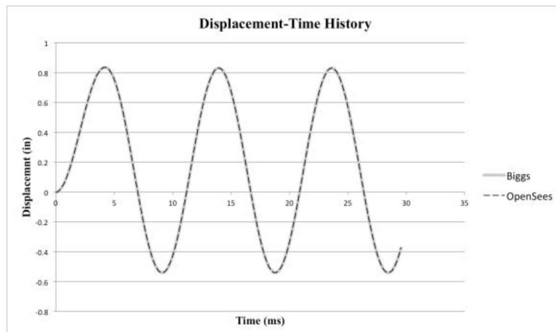
4 OPENSEES MODELLING EXAMPLE

As part of an initial study, a basic blast example, which could be solved analytically as is shown in Biggs [5], was modeled in OpenSees to begin the framework for various other incorporations.

The example is shown in Figure 3 and includes an 11 ft, W12x58 simply supported beam with applied approximate blast load of 250 psi peak pressure and 5 ms duration. The result from the OpenSees model is also shown on the right and is compared with the analytical solution from Biggs. In both cases the maximum displacement and maximum rebound displacement are identical at 0.84 in and -0.54 in. These values can be used to compute curvature and ductility and compared with various design guidelines.

The author is currently working on computations of the reaction forces, application of axial preload and applying gravity loads to check for collapse.





6 CONCLUSIONS

Advanced analysis tools for blast are greatly needed. To begin to rectify this issue, initial examples were carried out in OpenSees to match current analysis methods. It was shown that OpenSees can predict the response of components to blast loads via comparison to an analytical solution. A tremendous amount of future work, both numerically and experimentally, is needed to analyze the structural system as a whole and to account for localized behaviors.

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Faculty of Engineering, University of Porto

3-4 July 2014, Porto

An Integrated Computational Environment for Simulating Structures in Real Fires

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ABSTRACT

To protect the life and livelihoods of people in the built environment engineers are required to provide built infrastructure with a reasonably adequate level of resistance to the extremes of loads on structures resulting from natural and/or manmade hazards, such as earthquakes, tsunamis, hurricanes, fires etc. Given the complexity and uncertainty associated with these loads, traditional practice has often relied upon gross simplifications in representing them and enforced these representations through building codes and regulations, leading mostly to conservative designs. This however is not always the case as structural failures, where deficiencies of such prescriptive approaches can be identified as the cause, occur with an unnerving regularity. In recognition of this fact, there is an increasing consensus in the profession of structural engineering that the enforcement of sweeping simplified rules is perhaps not satisfactory in all cases. When the structure and the loadings expected to act upon is considered to be sufficiently complex and uncertain, engineers should be allowed, or even required, to consider alternative methods of engineering resistance delivering improved robustness (degree of safety against disproportionate failures) and resilience (the ability to recover rapidly from a disruptive event). This however is easier said than done, as alternative or the so-called performance-based engineering (PBE) approaches demand a much greater level of knowledge and understanding, supported by advanced computational tools for engineers to quantify as accurately as possible, the magnitudes of the expected

loading and the response of the structure (ideally also quantifying the key uncertainties associated with the demand and resistance).

This presentation will provide details of an integrated computational environment for simulating structures subjected to real fires, being developed at Edinburgh (in collaboration with partners at Tongji University in China). This tool is based upon the OpenSees software framework developed at University of California, Berkeley, the reasons for this choice will be outlined. An introduction to the key issues associated with computational modelling of structures subject to fires will be discussed, followed by examples of previous modelling work undertaken on other platforms and discussion of the deficiencies of those approaches. The vision for the new tool and its conceptual structure will then be presented followed by the progress so far with examples and a discussion of the ongoing work

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Steel4 – A Versatile Uniaxial Material Model for Cyclic Nonlinear Analysis of Steel-Based Elements

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SUMMARY

Steel4, a new material model for nonlinear dynamic analysis of steel-based anti-seismic solutions has been developed and implemented in the OpenSees environment. The paper highlights the main features of the highly customizable Steel4. It handles nonlinear kinematic and isotropic hardening and their combination; ultimate strength limit can be prescribed to provide more accurate material response at large deformations; it models asymmetric kinematic and isotropic hardening while preserving the Bauschinger effect in the material response; its memory of previous load history allows initial load cycles to influence future behaviour. The material has already been applied to modelling buckling restrained braces and conventional steel braces and showed promising results.

Key Words: *steel material model, cyclic nonlinear hardening, buckling restrained brace.*

1 INTRODUCTION AND OBJECTIVES

The exponential growth of available computational performance led to important changes in the nonlinear analysis of dissipative structural solutions in earthquake engineering. Nonlinear response history analysis (RHA) with finite element (FE) models is no longer reserved for researchers with access to supercomputers. Even probabilistic assessments based on thousands of such analyses can be performed on a stronger workstation within reasonable time. Due to concepts such as Performance Based Design these tools have already been applied by numerous practicing engineers in the past decade and their number is rapidly increasing.

However, computational resources are still insufficient for FE analyses on realistic numerical models of buildings made of solid elements. Structural behaviour is typically evaluated based on beam models often only in 2D. The numerical material model plays an important role in result accuracy, because in many cases it is solely responsible for reproducing the nonlinear response of the modelled element. Despite their importance, a large number of studies apply rather crude material models. These typically bilinear approaches fail to capture the behaviour of even simple steel braces with sufficient accuracy under irregular cyclic loading such as seismic excitation. Note that material nonlinearity in itself does not ensure realistic modelling. It is proper calibration and sufficiently advanced material capabilities that can lead to realistic structural response in a numerical environment.

The authors are not aware of any numerical material model in popular FE environments that provides straightforward calibration, advanced behaviour and computational efficiency simultaneously. Even OpenSees (OS) with its large material library does not have a solution for this problem, because each advanced material is typically designed for a special purpose and it is difficult to find one that works well for a new problem. Calibration can also prove challenging as the user seeks to follow more advanced behaviour.

The authors decided to improve on this situation by development of a general purpose material model for steel-based elements. The model is implemented in the framework of OS. The material shall be capable of accurate modelling of any steel-based element for RHA. Improved accuracy shall not automatically lead to unreasonable complexity. Besides accuracy efficiency is also important. The material is based on a simple nonlinear uniaxial phenomenological formulation originally developed by Giuffré, Menegotto and Pinto. *Steel02*, an extension of their material is already available in OS. The new material is proposed to have the name *Steel4* to acknowledge its origins.

2 FEATURES OF STEEL4

Steel4 provides a highly customizable model to nonlinear cyclic hardening behaviour. The material model is controlled by a total of 26 parameters. 25 of these are optional, thus the response can be tailored to the needs of the user. The following paragraphs give a brief overview of the capabilities of this new material model and explain the application of its parameters. *Figs. 1* and *2* illustrate each parameter involved in modelling different forms of kinematic and isotropic hardening, respectively.

The only non-optional parameter is the initial stiffness (E_0) of the material. Providing solely this value creates the simplest possible linear elastic material. This material is identical to *Elastic* in OS. If f_y and b are also specified, a bilinear kinematic hardening model will be created that resembles *Steel01*. Three additional parameters are required to control a nonlinear transition from elastic to hardening behaviour: R_0, r_1, and r_2. The role of each parameter is the same as in *Steel02*.

Ultimate strength is specified by f_u. Material behaviour at that stress level becomes perfectly plastic. The transition from hardening behaviour is controlled by R_u. Introduction of ultimate strength in the material model significantly improves response accuracy for large amplitude cyclic loading [1]. Special care has been taken to ensure appropriate modelling of the Bauschinger effect (Fig.1).

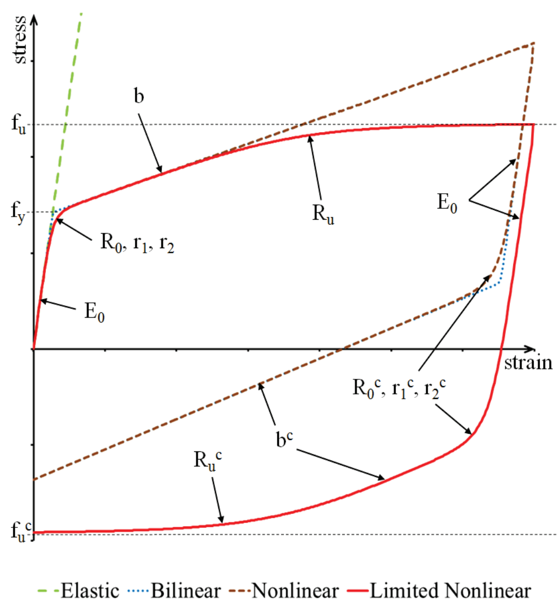

Fig. 1 Behaviour of *Steel4* with different kinematic hardening setups under cyclic loading

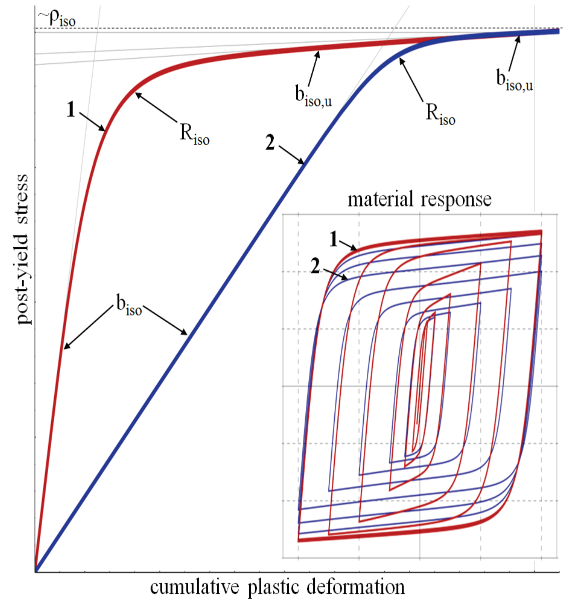

Fig. 2 Response of two different isotropic hardening setups of *Steel4* under cyclic loading

Isotropic hardening can also be modelled by *Steel4* (Fig. 2). The development of isotropic hardening is controlled by b_{iso}, R_{iso} and $b_{iso,u}$. If the last two parameters are omitted, a bilinear isotropic hardening model will be created. Including all three parameters allows the user to model custom hardening saturation as shown in Fig. 2. Isotropic hardening is a function of cumulative plastic deformation of the material. The extent of hardening can be controlled by ρ_{iso}. The yield plateau after first yield can also be included: l_y describes the length of the plateau in units of yield deformation ε_y. This feature is especially useful for numerical analysis of monotonic loading.

Steel4 is capable of modelling elements that have different hardening characteristics under tension and compression. The authors refer to this phenomenon as asymmetric hardening. When such behaviour is desired, additional parameters for hardening under compression need to be specified. When the asymmetric hardening feature is used the basic parameters presented above are used to describe hardening under tension. Under compression kinematic hardening is controlled by b^c, R_0^c, r_1^c and r_2^c; isotropic hardening by b_{iso}^c, R_{iso}^c, ρ_{iso}^c, $b_{iso,u}^c$; while the ultimate strength limit is defined by f_u^c and R_u^c. The Bauschinger effect is taken into account even if a material with asymmetric hardening properties has limited strength capacity in both directions (Figs. 1 and 2).

Initial stress modelling developed for *Steel02* is preserved in *Steel4* and specified by the same σ_{init} parameter. The final *LHM* parameter is a boolean for controlling the load history memory capability of the material. When *LHM* is on, the material model stores information on all previous load cycles during the loading process. This information is currently used to enforce the response to converge to previous cycles. This approach can efficiently avert a typical error in *Steel02* that occurs after abrupt unloading-reloading sequences and leads to overestimation of the stress response of the material.

Beam models using Steel4 have been verified against advanced solid and shell models developed in ANSYS [3]. Detailed information on the *Steel4* material and a short summary of the verification results is available in [2].

3 APPLICATIONS, CONCLUDING REMARKS

Development of *Steel4* is part of a research on probabilistic seismic assessment of frames with Buckling Restrained Braces (BRB) [2]. Therefore, the authors already have considerable experience in application of *Steel4* to modelling BRB elements. This experience confirms that the main advantage of *Steel4* compared to other numerical material models is that its parameters can be directly associated with certain properties of the element or material under examination. Therefore, calibration is simplified, because the majority of parameters are pre-defined by data from material test results and geometric properties of the specimens.

The authors were able to create a set of simple rules for selecting *Steel4* parameters based only on geometric data and steel material test results for a certain type of BRB elements [2]. The resulting material models were verified using 15 experimental test results and showed excellent performance. This implies that the created model is able to predict the response of any BRB as long as it is made by the same manufacturing process (i.e. has the same basic configuration, but might have different geometric proportions or steel material).

Steel4 has also been applied to steel-only BRB elements [4] in an ongoing research. Calibrated parameters give a better understanding of the influence of certain geometric parameters on the nonlinear cyclic behaviour of the element. Besides BRBs, application of *Steel4* to conventional steel frames has also been investigated. A comparative analysis concluded its superiority over simpler materials such as *Steel02* and *Steel01* [3].

Promising results so far have shown that *Steel4* improves modelling accuracy without increasing the complexity of calibration or model development. The authors hope that *Steel4* can be part of the official OS release in the near future and facilitate structural modelling for many earthquake engineering professionals.

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OpenSees as an Engine for Web-Based Applications

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SUMMARY

The emergence of cloud technologies is gradually introducing new paradigms in every engineering field. The availability of a wide variety of open-source technologies is also promoting the development of new services that are exclusively provided through the internet. In the case of structural engineering, it is now possible to perform advanced structural analysis computations using powerful HPC clusters available on the internet. At the same time, and due to the open-source nature of OpenSees, it is now viable to develop new structural engineering tools, in the form of web services, which are exclusively provided through the web. This paper describes the implementation of a web service and a web application prototype that were developed to aid structural engineers in the analysis and design of composite steel-concrete cross-sections. In addition to providing easy access to cross-section geometrical and mechanical properties, the application also performs the calculation of axial force-bending moment (N-M) interaction curves, which are generated in real-time using the OpenSees framework.

Key Words: *composite steel-concrete sections, web services, web applications.*

1 INTRODUCTION

The use of web-based technologies is becoming a standard in every engineering domain. In spite of the advantages associated with these technologies, their adoption for civil engineering applications has been slow. Recently, Barros *et al.* [1] discussed the use of web-services in a civil engineering context and presented an implementation for a simple structural engineering problem.

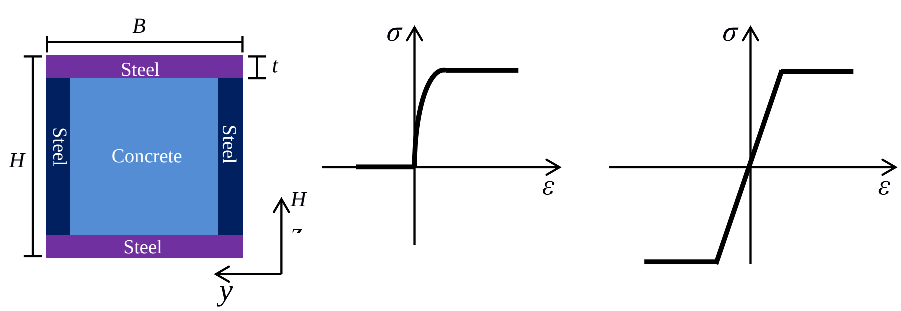
The benefits of using these technologies are extremely relevant and can be potentiated when combined with the development of advanced software applications for the solution of complex engineering problems.

Axial force-bending moment (N-M) interaction curves are widely used in structural design. Whilst their derivation is a trivial problem for simple cross-sections subjected to uniaxial bending, that is not the case for cross-sections characterized by complex geometries and/or multiple materials subjected to either uniaxial or biaxial bending conditions. Concrete filled steel tubes (CFST) are an example of such type of cross-sections.

This communication briefly describes the development of an OpenSees model for the calculation of axial force-bending moment (N-M) interaction curves of composite steel-concrete cross-sections. It is also presented the implementation of a web service which allows internet users to utilize the model in different computing environments. An example is provided in which the web service is used in the context of a web application.

2 ANALYSIS OF COMPOSITE CROSS-SECTIONS IN OPENSEES

A simple way to derive the N-M interaction curves of a given cross-sections is to perform a fibre-based nonlinear analysis. In this work OpenSees was used to model both rectangular and circular cross-sections of concrete filled steel tubes. Figure 1(a) illustrates a rectangular cross-section of this type with external width *B* and external height *H*. The steel tube thickness is *t*, considered to be equal for all the plates. The analysis of the cross-section is performed with a zero-length element and with a FiberSection object available in OpenSees. The concrete core is modelled as a set of fibres with the **patch rect** command. The steel tube is divided into 4 small rectangular sections and modelled with the same **patch rect** command. As for the materials, the **Concrete01** command, which simulates an uniaxial material with zero tensile stress, is used to model the concrete. The plastic range of the stress-strain curve is set as perfectly plastic to be consistent with the simplified model of Eurocode 2 (Figure 1(b)). The steel is modelled as uniaxial material with command **Steel01**. The strain-hardening ratio (b) is set to 0 to simulate an idealized elasto-plastic behaviour (Figure 1(c)).



(a) CFST cross-section (b) Concrete constitute law (c) Steel constitute law

Figure 1. Section modelling and steel properties definition

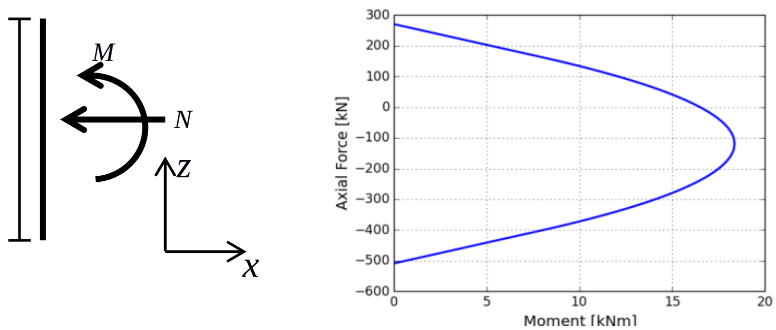
As both the concrete and steel are modelled as uniaxial materials with idealized elasto-plastic behaviour, the compressive (N_{comp}) and tensile (N_{ten}) resistances of the cross-sections can be calculated as follows:

$$N_{comp} = f_c A_c + f_y A_s \tag{2.1}$$

$$N_{ten} = f_y A_s \tag{2.2}$$

where f_c is the compressive stress resistance of the concrete and f_y is the yield stress of the steel. A_c and A_s represent the steel and concrete areas, respectively.

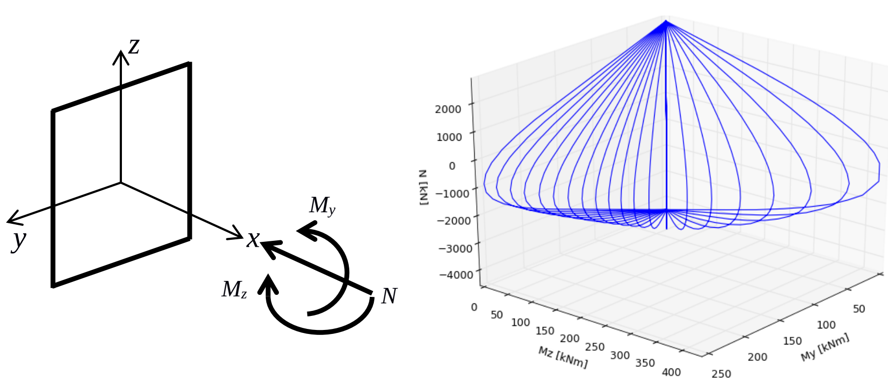
In order to get a full interaction curve with the axial load N ranging from N_{ten} and N_{comp}, different levels of axial loads, which are uniformly distributed in the interval [$-N_{com}$,N_{ten}], are considered. For each axial load level, N_i, a moment-curvature analysis is conducted in OpenSees by firstly applying the axial load and then an increasing bending moment under rotation control. The maximum bending moment, M_i, is recorded, allowing in this way to get one point (M_i, N_i) of the interaction curve. A Python script was developed to perform this sequence of analysis automatically. Figure 2 illustrates the interaction curve of a CFST cross-section under uniaxial bending.



(b) *N-M* interaction curve

Figure 2. *N-M* interaction curve of a CFST cross-section

For biaxial bending, the method to get the interaction surface (N-M_y-M_z) is similar to the method for obtaining the *N-M* curve for uniaxial bending. The 3D surface can be seen as an assemblage of several N-M interaction curves with varying M_y/M_z ratios. With this ratio ranging from 0 to an infinitely large value, all the points on the 3D surface can be derived (Figure 3).



(a) Section forces

(b) N-M_y-M_z interaction surface

Figure 3. Interaction surface of a CFST cross-section

3 IMPLEMENTATION OF THE WEB SERVICE

A web service has been developed to expose the tool developed in the previous section on the internet. This has been achieved by implementing the Python script that calls OpenSees in the OpenG API framework described in [1]. The web service responds to client requests that are transmitted to the web server using the HTTP protocol. The URL contains all the information related to the request, namely the cross-section and material properties. Although this is not the only way to establish a communication between a client and a web service, it is widely adopted by the internet community. The definition of the structure of the URL is therefore of paramount importance. In this context, a REST approach has been adopted in order to define URL structures that are intuitive to the user. The data is provided in a JSON format which can be readily used in dynamic web applications. Figure 4 shows a screenshot of the Flange+Web web application [1] (it is still a beta version) that provides the user with properties of steel and composite cross-section. The application is still in a beta status and at this stage it does not allow the representation of 3D interaction surfaces but only N-M interaction curves for uniaxial weak and major axis bending loading cases.

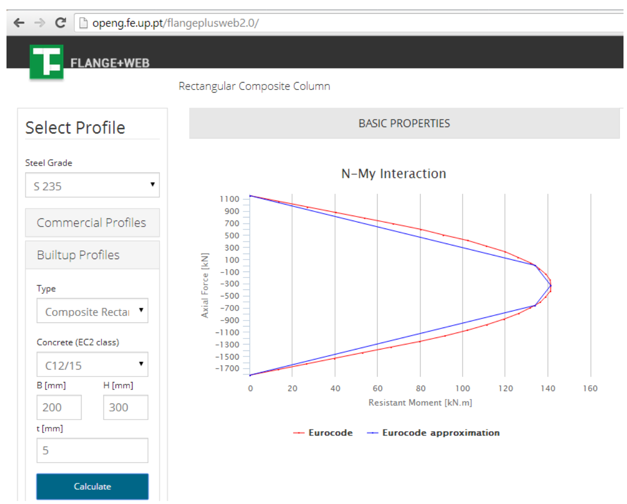


Figure 4. Interaction surface of a CFST cross-section

4 CONCLUSIONS

This communication described the development of a tool for the calculation of N-M interaction curves of composite steel-concrete cross-sections. The tool was implemented in a web API and hence it can be accessed through the internet. A web application was developed to demonstrate one of the various uses of this tool and the advantages of API applications in civil engineering.

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Numerical simulations of liquefaction phenomena after Emilia Romagna (20 May 2012) earthquake

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SUMMARY

Soil liquefaction has been observed worldwide during earthquakes with induced effects responsible for damage, disruption of function and considerable replacement expenses for structures. The May 20, 2012 M5.9 shock in Emilia Romagna, Italy is one example of moderate earthquakes yielding extensive liquefaction-related phenomena. The paper aims at reproducing on-site evidences comparing the most credited current approaches to estimate liquefaction risk with numerical computations. The study adopts a FE computational interface (OPENSEES PL) implemented in OpenSees and able to analyse the earthquake-induced three-dimensional pore pressure generation.

Key Words: *earthquake engineering, liquefaction, current methodology, numerical simulations, OpenSees PL*

1 INTRODUCTION

The May 20, 2012 M5.9 shock in Emilia Romagna, Italy is one example of moderate earthquakes yielding extensive liquefaction-related phenomena, such as sand boils, ground failures, intense ejection of sand and water. Locally, the observed lateral spreading and foundation settlements affected the stability of buildings and impacted social life inducing valuable lessons on liquefaction risk assessment. The most affected area is geologically characterised by sandy and silty saturated layers of ancient river beds (Po, Panaro and Reno) of gentle slopes of a few degrees. The most significant phenomena of liquefaction have been observed in San Carlo settlement (located about 17 km from the epicentre), under the Municipality of Sant'Agostino, near Ferrara, where large horizontal and vertical displacements induced local and global instability to the buildings and lifelines unserviceability. For more details, see Crespellani et al. 2012, [1]. The paper aims at reproducing these on-site evidences comparing the most credited current approach to estimate the liquefaction risk with numerical computations. In this regards, current methodologies verify liquefaction risk in terms of total pressure, without a control on excess pore pressure. They generally refer to in situ tests results and they are based on empirical relationships. On the other hand, Finite-Element (FE) simulations are increasingly providing a versatile environment allowing more detailed information on effective damage and economic assessment. In this regards, the study adopts a FE computational interface (OPENSEES PL) implemented in OpenSees (Mazzoni et al. 2006, [6]).

2 CURRENT METHODOLOGY

Traditional methodologies consist in defining liquefaction risk adopting very few (at least one) parameters from observed events and laboratory tests. Nowadays a great number of existing methods can be adopted and they are generally built up with some significant parameters taken from geological and geotechnical recognitions, such as layer age, origin, water depth. Some of these methods were transferred inside international codes such as Eurocode 8. Current methodologies verify the risk for the soil to be subjected to liquefaction without a direct control on excess pore pressure. They refer only to few descriptive parameters based on historic knowledge and on behaviour analysis results of laboratory cycling test under controlled conditions. In this regards, they should be used only in preliminary studies aimed at predicting liquefaction in qualitative terms. For more details, see Forcellini et al. 2013, [4].

3 NUMERICAL METHODOLOGY

In this paper credited non-linear theories are adopting taking into account appropriate loading-unloading flow rules as to reproduce the observed strong dilation tendency and resulting increase in cyclic shear stiffness and strength (Yang et al. 2003, [7] and Elgamal et al. 2003, [3]). Even if based on these theories, the models have the main advantage to be built up with the most common-used geotechnical parameters. In particular, two different models for cohesionless and for cohesive soils were considered. The first model for cohesionless materials is developed within the framework of multi-yield-surface plasticity. Clay soils are modelled as nonlinear hysteretic materials with a Von Mises multi-surface kinematic plasticity model. The adopted parameters were calibrated through an identification analysis taking into account non-linear liquefaction-induced behaviors, for more details, see Yang et al. 2003, [7] and Elgamal et al. 2003, [3]. OpenSees PL used in this study, originally calibrated for pile analyses (Elgamal et al., 2009, [2]), was modified in order to take into account free field conditions. The interface simplifies the 3D spatial soil domain, boundary conditions and input seismic excitation definition with convenient post-processing and graphical visualization of analysis results including deformed ground response time histories. The possibility to simulate wave propagation adopting realistic boundaries is of particular importance and significance in order to realistically reproduce liquefaction response. In particular, recordings were taken from Mirandola (MRN) station, the closest station (about 13.4 km from the epicentre of May 20, 2012 shock). The model consists of a 3D 20 x 20 m, 90 m high model (Figure 1), representing free field conditions and modeled with periodic boundary conditions on account of symmetry (at any spatial location displacement degrees of freedom of the left and right boundary nodes were tied together both longitudinally and vertically using the penalty method). Thus, base and lateral boundaries were modelled to be impervious, as to represent a small section of a presumably infinite (or at least very large) soil domain by allowing the seismic energy to be removed from the site itself. For more details, see Elgamal et al. 2009, [2], and Forcellini and Tarantino, 2012, [5] and Forcellini et al. 2013, [4].

4 RESULTS

In this section free field results are shown in terms of excess pore pressure and longitudinal displacements time histories. In particular, Figure 2 shows that excess pore pressure reaches the peak value around 120 kPa at the base of sand strata (13 m) and starts rapidly decreasing after about 15 s (at the same level as 8m) and then fully dissipates at 40 s. The results show that the pick value of pore pressure concentrates between 8.00 m and 13.00 m, in correspondence of the intermediate sand layers that result to be the principal cause of liquefaction induced effects. This result is confirmed taking into consideration lateral displacements (Figure 3). At 8.00 m and 0.00 m depth displacements, no big difference in maximum values (around 32 - 35 cm) can be seen, while at 13.00 m depth, the final value is around 6 cm. This enforces the role of intermediate sand layers in liquefaction induced lateral spreading generation, since the longitudinal displacements totally concentrate between 8.00 m and 13.00 m. Permanent displacements at the surface (around 35 cm) are comparable to those measured during recognitions in San Carlo (for more details, see Crespellari et al. 2012, [1]).

5 CONCLUSIONS

The paper presents a computational model of free field response (adopting OPENSEES PL) that numerically reproduces Italian Emilia-Romagna Earthquakes (May 2012). Response underlines vulnerability of such submerged 3D system in terms of pore pressure generations and lateral displacements, confirmed by site recognitions. Results verify the role of intermediate sand layers in liquefaction induced lateral spreading generation. In particular, this numerical study can quantify soil performance to liquefaction induced effects using metrics that are of immediate use for both pre-earthquake and post-earthquake risk assessment analyses, otherwise impossible to be computed by traditional methodologies.

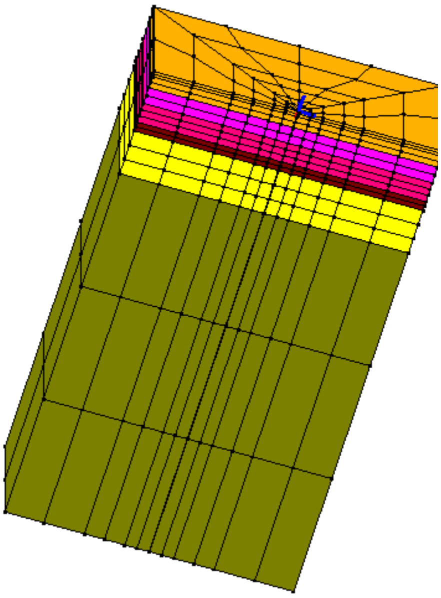

Figure 1 Free-field 3D model

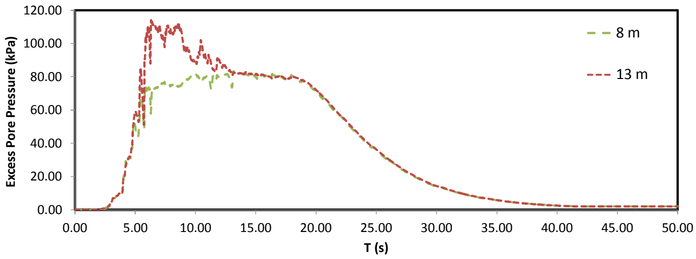

Figure 2 Excess Pore Pressure at 13.00 m and 8.00 m depth

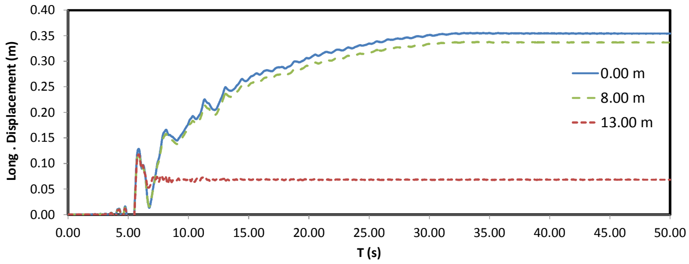

Figure 3 Long. displacement at 13.00 m, 8.00 m and 0.00 m depth

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Seismic Loss and Downtime Estimates of Existing Tall Buildings and Strategies for Increased Resilience

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SUMMARY

Tall buildings play an important role in the socio-economic activity of major metropolitan areas. The resilience of these structures is critical to ensure a successful recovery after major disasters. Events such as the Canterbury earthquake in 2011 have highlighted the impact of poor performing buildings on the business continuity of downtown districts, where tall buildings are typically clustered together. Following the 2011 earthquake, Christchurch's Central Business District (CBD) red zone covered a significant area of the city and more than 60% of the businesses were displaced (CERC 2012).

Until the introduction of Performance Based Seismic Design (PBSD) in the 1990s, buildings were designed using conventional building codes, which follow a prescriptive force-based approach based on the first mode translational response of the structure (FEMA 2006). Researchers and engineers have raised concerns that the prescriptive approach of building codes is not suitable for tall building design due to the significant contribution of higher mode effects (PEER 2010a). As a result of these shortcomings, several jurisdictions in areas of high seismicity throughout the Unites States (e.g. Los Angeles and San Francisco) have adopted a PBSD approach for the design of new tall buildings. While new designs follow a more adequate approach, little is known about the seismic performance of older existing tall buildings that were designed prior to the adoption of PBSD (Almufti et al. 2012).

This paper presents an assessment of the seismic performance of existing tall buildings in a case study city, San Francisco, where an archetype tall building is designed based on an inventory of the existing tall building stock. Non-Linear Response History Analysis (NLRHA) are conducted with ground motions representative of the design earthquake hazard level defined in current building codes, with explicit consideration of near-fault directivity effects. In order to influence decision making, performance is reported as the expected consequences in terms of direct economic losses and downtime. Once the performance of the archetype building is assessed, a range of structural and non-structural enhancements are explored for enhanced performance as well as mitigation measures for increased resilience.

Key Words: *seismic losses, downtime, retrofit, tall buildings.*

1 INTRODUCTION

Engineering seismologists, seismic, structural and geotechnical engineers have demonstrated that through Performance Based Earthquake Engineering (PBEE), detailed assessments of individual building performance can be conducted through complex NLRHA. These tools can be used for new design or assessment of existing buildings in order to ensure adequate performance under seismic events of a specified return period. Furthermore, estimates of the monetary losses associated to the performance of the building can be conducted through tools such as Performance Assessment Calculation Tool (PACT) (ATC 2012). Recent developments (Almufti and Willford 2013) have also been made in order to provide a direct measure of resilience through a downtime assessment methodology, which identifies the likely causes of downtime such that these can be mitigated to achieve a more resilient design.

This paper assesses the performance of existing tall buildings in a case study city, San Francisco, through the development of a database of the existing tall building stock, the design of a representative archetype building, performance assessment via NLRHA with ground motions representative of the design earthquake hazard level and estimation of associated direct economic losses and downtime. Once the performance of the archetype building is assessed, a number of strategies for increased resilience are proposed including structural retrofits, non-structural enhancements and mitigation measures. The work conducted to date provides a good understanding of the expected performance of an archetype tall building in the city of San Francisco under a design level earthquake. It also illustrates that a number of measures can be implemented to increase the resilience of these existing buildings.

2 MAIN BODY

The first step of the methodology is to develop a database of the existing tall building stock for the city of interest. This database includes information about each building, such as address, year built, number of storeys, lateral resisting system, whether retrofit or upgrades have been conducted, building use, etc. When possible, existing building drawings are reviewed. However, such reviews are not always feasible due to the lack records or public access to these. The purpose of the database is to develop archetype building designs that are representative of the existing tall building stock. These archetype buildings are developed by disaggregating the existing tall building database in order to identify trends. Archetype buildings focus on the following main variables: year of construction (to identify relevant building codes for design and typical construction of the time), lateral resisting system type and number of storeys. These key variables influence building performance and are therefore critical in developing archetype buildings for design.

The second step is to visualize the geographical location of the existing tall building database in a Geographical Information System (GIS) tool in order to identify representative sites. The purpose of identifying representative sites is to conduct Probabilistic Seismic Hazard Analysis (PSHA). Soil data at the representative sites is of relevance as it influences the site specific hazard. This paper presents the results of an intensity based assessment (single intensity level) under a design level earthquake. An array of ground motions needs to be selected, scaled and modified at the selected intensity level. These motions are then utilized to evaluate the seismic demand of buildings through NLRHA simulations.

The third step is to develop a numerical model for the archetype building in order to conduct NLRHA. The analytical models are three-dimensional and represent all components and force and deformation characteristics that significantly affect the seismic demands. Element properties are based on expected values of strength (PEER 2010b) to capture anticipated behaviour.

The fourth step is to develop a building performance model - defined as a model to assess the probability of earthquake losses and downtime. The building performance model includes all structural and non-structural components that are susceptible to earthquake damage. It enables calculation of direct economic earthquake losses and downtime estimates.

Based on the structural performance, loss assessment and downtime estimates for each archetype building, strategies for increased resilience are developed. These strategies include structural retrofits, adoption of non-structural building components that are more resilient to earthquake damage or a combination of these. Analytical models for structural retrofit strategies are developed and re-analysed to quantify the reduction in seismic demands. Revised building performance models are developed including all enhanced non-structural components and the loss assessments are revisited to quantify the reduction in losses associated with these components. Similarly, downtime estimates are re-examined in order to quantify the impact of these strategies on the resilience of the building

The city of San Francisco is selected as a case study due to a number of factors. San Francisco is one of the most seismically vulnerable cities in the world due to its large number of older buildings and proximity to major active faults. The city has a large number of existing tall buildings that were designed from the 1960s to the 1990s following prescriptive code guidelines that do not provide an explicit understanding of building performance in earthquakes. Additionally, past earthquake events, such as the Northridge earthquake in 1994, highlighted deficiencies in the moment resisting connections of steel Moment Resisting Frames (MRF), which is a lateral resisting system adopted by many of the existing tall buildings in the city.

3 CONCLUSIONS

An assessment of the seismic performance of existing tall buildings in a case study city, San Francisco, is presented. The assessment is conducted through the development of a database of the existing tall building stock, the design of a representative archetype building, performance assessment via NLRHA with ground motions representative of the design earthquake hazard level and estimation of associated direct economic losses and downtime. Expected losses are in the order of 34% building replacement cost. Expected downtime for functional recovery is in the order of 87 weeks. A number of strategies for increased resilience have been presented to enable a reduction in losses to 3% building replacement cost and downtime for functional recovery of 1 day.

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Deterioration Modeling of Structural Members Subjected to Cyclic Loading Using Concentrated Plastic Hinge and Finite-length Plastic-Hinge Models

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SUMMARY

Performance assessment of structures under severe earthquakes is highly dependent on the behavior simulation of structural members under large cyclic deformations. However, this behavior is extremely complex, and can only be modelled by reproducing, at a section level, the behavior observed experimentally. Thus, the use of empirically calibrated moment-rotation models that account for strength and stiffness deterioration of structural members is thus fundamental. These deterioration models are typically employed to define the flexural behaviour of zero-length springs in a concentrated plastic hinge formulation. Otherwise, a calibration procedure is required when they are used to represent the moment-curvature ($M - \chi$) behavior in distributed plasticity formulations because the resulting moment-rotation ($M - \theta$) response depends on the element integration method. A plastic hinge integration method for using deterioration models in force-based elements is developed and validated using flexural stiffness modifications parameters to recover the exact solution for linear problems while ensuring objective softening response. With this approach, moment-rotation models that account for strength and stiffness deterioration can be applied in conjunction with force-based plastic hinge beam-column elements to support collapse prediction without increased modeling complexity.

Key Words: *Component deterioration; Force-based finite elements; Finite-length plastic hinge; Plastic hinge calibration; Steel and Cyclic loading.*

1 Background

Performance-based seismic design and assessment requires accurate nonlinear finite element models that can capture the full range of structural response associated with various performance targets. In the development of realistic finite element models, modes of strength and stiffness deterioration attributable to damage accumulation that could lead to local or global collapse need to be identified. However, simulating the behavior of structural member under these extreme loading conditions is extremely complex, and can only be accomplished by reproducing, at a section level, the behavior observed experimentally. Thus, the use of empirically calibrated moment-rotation models that account for strength and stiffness deterioration of structural members is paramount in evaluating the performance of steel structures prone to collapse under seismic loading.

The main objective of this paper is to present the implementation details for Modified Medina-Ibarra-Krawinkler Deterioration (ModIMK) models (Lignos and Krawinkler 2012) for use in concentrated plastic hinge models as well as in finite-length plastic hinge models. Furthermore, a calibration procedure for the finite-length procedure is proposed. The ModIMK models is a complex and general model, which accounts for six different deterioration mechanisms, and for that reason, ideal to demonstrate the applicability of the proposed models and for use in collapse structural analysis.

2 Simulating member behavior

2.1 Concentrated plastic hinge models

The empirically deduced deterioration models are typically assigned to define zero-length springs in concentrated plastic hinge (CPH) models (Giberson 1969). In these models, nonlinear zero-length moment rotation springs are located at both ends of a linear-elastic beam-column element. If a CPH model is used in the development of a structural model, moment-rotation relationships directly obtained from experimental tests can be employed to define the nonlinear zero-length springs that control element flexural response. The correct linear-elastic solution for the entire element is only obtained if the end rotational springs are approximated as rigid-plastic. This is usually achieved using an ad-hoc stiffness modifier parameter, n, for the zero-length springs.

Since the elastic stiffness of the moment-rotation relationship needs to be assigned a large value to make it rigid-plastic, the direct application of the constitutive law to the zero-length plastic hinges requires the development of an algorithm for updating, during the analysis, the spring stiffness. This algorithm is presented in this work. Without the use of such algorithm results obtained using CPH models cannot correctly include modes of deterioration such as unloading stiffness deterioration or reloading stiffness deterioration.

2.2 Finite-length plastic-hinge models

An alternative formulation for simulating member response is to use distributed plasticity finite elements. Based on this formulation, alternative approaches have been proposed by different authors, trying among others, to limit the effects of localization phenomena (Coleman and Spacone 2001). Of those, the finite length plastic hinge element, proposed by Scott and Fenves (2006) has shown some significant advantages. When compared to the concentrated plasticity approach, this model has been shown to be advantageous, namely in what concerns to modeling effort, computational cost, and clear separation between member and connection nonlinearity. It should be noted that the finite length plastic hinge model is an efficient distributed plasticity formulation with designated hinge zones at the member ends. Cross sections in the inelastic hinge zones are characterized through either nonlinear moment-curvature relationships or explicit fiber-section integrations that enforce the assumption that plane sections remain plane. These models result in more realistic modeling of the spread of yielding than the concentrated hinges, while the finite hinge length facilitates calculation of hinge rotations.

When finite-length plastic-hinge (FLPH) elements are used, a calibration procedure is required, namely when moment-rotation models are used to represent the section moment-curvature ($M-\chi$) behaviour because the resulting moment-rotation ($M-\theta$) response depends on the element integration method. A newly proposed calibration procedure (Ribeiro et al., 2014) is employed in this work. This calibration procedure makes use of flexural stiffness modifications parameters to recover the exact solution for linear problems while ensuring objective softening response. To guarantee accurate results in both the linear and nonlinear range of response, the flexural stiffness modification parameters are computed at the beginning of the analysis as a function of the user-specified plastic hinge length L_p. In Fig. 1 is shown the correspondent integration scheme, where the locations and section stiffness matrixes of all integration points are represented.

In addition to the afore mentioned advantages of the FLPH models over the traditional CPH models, it is worth noting that the use of FLPH models do not need any algorithm to update hinge stiffness as these models do not require any parameter n to be used. However, careful definition of the deterioration parameters should be performed since the moment-curvature relationship that has been assigned to the hinge section relates to the empirically deduced moment-rotation relationship though the plastic hinge length L_p, as expressed in Eq. 1.

$$\begin{cases} k_{M-\chi} = k_{M-\theta} \times L_p \\ \chi = \theta / L_p \end{cases} \tag{1}$$

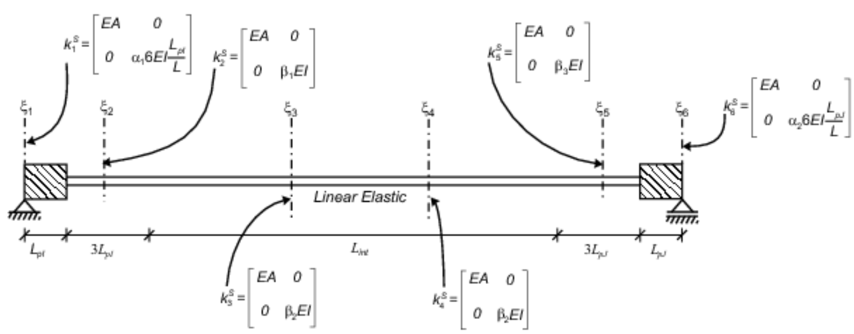

Figure 1. Integration scheme with section flexural stiffness modification parameters (Ribeiro et al., 2014)

3 Numerical example

A set of examples analyzing simple structures subject to cyclic pushover loadings is presented to evaluate the accuracy of the proposed implementation, as well as, its numerical stability. The formulation proposed was implemented in a modified version of the Open System for Earthquake Engineering Simulation (OpenSees, v2.4.3, r5695) framework. All examples consist of a beam-column finite-element basic system (simply-supported beam) subjected to end moments applied under controlled nodal end-rotations. Different Mod IMK nonlinear material models describing the hysteretic behavior of the beam are used. The examples and analysis shown herein are used for code verification and validation purposes.

The implementations of the FLPH model and CPH model (CPH Updated) described in this paper are considered for all cases. For comparison purposes, results from application of the direct implementation of the moment-rotation relationships in a CPH model (CPH Original), i.e. without using any algorithms that allows for correctly update the stiffness of the model, are also presented.

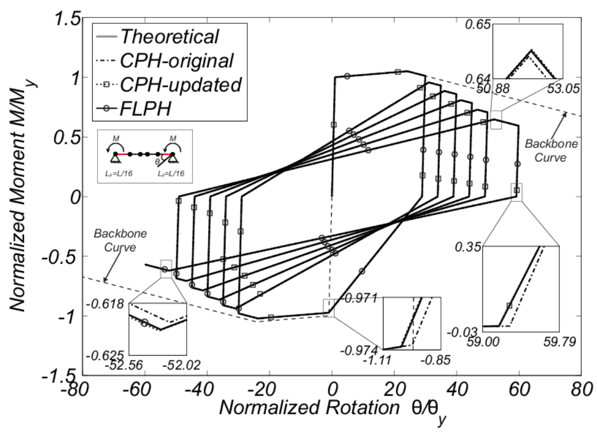

Figure 2. Cyclic pushover using the ModIMK Peak-Oriented model

4 Conclusions

Results obtained show that: (i) accurate results can be achieved either by using FLPH models or CPH models, since the proposed algorithm is used if the second models are employed, (ii) a significant reduction in model complexity is obtained when FLPH models are employed; (iii) the implementation procedure for FLPH models is significantly simpler than that required for the CPH models and the use of ad-hoc parameters simulating rigid plastic behavior can be avoided.

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Application of OpenSees in Reliability-Based Design Optimization of Structures.

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SUMMARY

Key Words: Reliability Based Design Optimization, Structural Reliability.

1. INTRODUCTION

Design optimization has undergone a substantial progress. Commercial finite element codes have recently added optimization capabilities. However, most of these developments deal only with deterministic parameters. Uncertainties are inherent in design variables and parameters such as material properties, loading and geometry parameters as well as the mathematical model of the system and it is necessary to consider these types of uncertainties in the design of any engineering system to ensure safety and quality. Traditionally, these uncertainties have been considered through partial safety factors in structural optimization methods. These partial safety factors are established in structural design codes, such as Eurocodes or the specific national building code for each country. These methods are named "semiprobabilistic" because uncertainties of the design variables are considered implicitly. However, safety factors do not provide a quantitative measure of the safety margin in design and are not quantitatively linked to the influence of different design variables and their uncertainties on the overall system performance. Therefore, it is crucial to account for uncertainties explicitly in structural design.

The process of design optimization enhanced by the addition of reliability constraints is referred as Reliability-Based Design Optimization (RBDO) or Reliabilty-Based Structural Optimization. A large research effort has been carried out about RBDO methods during the last thirty years. The main objective of this effort has been to reduce the large computational effort needed to obtain an optimum design that verifies reliability constraints.

"Finite Element Reliability Using Matlab" (FERUM) is a Matlab Toolbox for Reliability Analysis. Recently, new functions have been added to implement gradient-based RBDO methods. These functions can solve analytical and structural problems. This new functions form a specific toolbox named FERUM-RBDO [1]. Methods implemented in this toolbox are grouped in three wide groups: Double Loop methods, Decoupled or Sequential methods and Single Loop methods. Double Loop methods are well known and can be classified as RIA (Reliability Index Approach) based methods and PMA (Performance Measure Approach) based methods.

RBDO-FERUM Toolbox is based on software FERUM 3.1, developed by the Civil Engineering Department in University of California at Berkeley. However, finite element tools implemented in FERUM are limited. Because that, it is necessary to connect RBDO-FERUM with a powerful finite element analysis software.
In the other hand, design projects where it is worth to consider reliability constraints are large structures supporting extreme natural actions like large wind loads, earthquakes, wave loads, etc. In these cases linear structural analysis could be very inaccurate because material and geometrical nonlinearities occur. These reasons have motivated that OpenSees were chosen as the finite element software to use.

OpenSees, the Open System for Earthquake Engineering Simulation, is an object-oriented, open source software framework created at the Pacific Earthquake Engineering (PEER) Center. It allows users to create finite element applications for simulating the response of structural and geotechnical systems subjected to earthquakes. OpenSees provides also a great range of capabilities like nonlinear analysis, structural dynamic analysis, pushover analysis, time-history analysis, etc. Some sensitivity and reliability commands are available and have been used in this work [2].

2. IMPLEMENTATION

First part of this work is to implement the double loop RBDO methods combining RBDO-FERUM and OpenSees. Source files writing for the OpenSees interpreter contains commands to compute the terms needed in the outer optimization loop for double loop methods. In the case of RIA based double loop method OpenSees can provide reliability indexes corresponding to limit state functions or performance functions and the gradients of these reliability indexes with respect to mean values and standard values of random variables modeling uncertain structural parameters as loads, material properties or geometric dimensions.
When PMA based double loop methods are used, OpenSees can provide the values of the performance functions and the gradients of the performance functions with respect to random variables.
Communications between RBDO-FERUM and OpenSees are carried out by writing and reading *.tcl and data files with results. Figure 1 and 2 describe communications between outer loop and inner loop for RIA and PMA respectively.

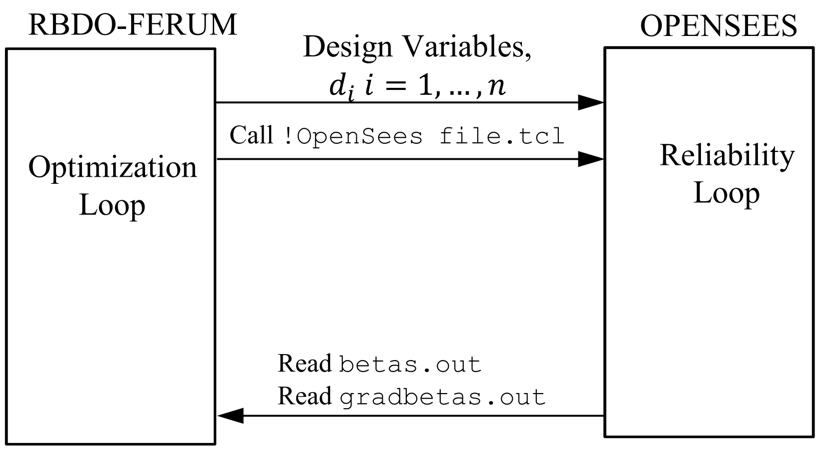

Figure 1. RBDO RIA-based double loop method

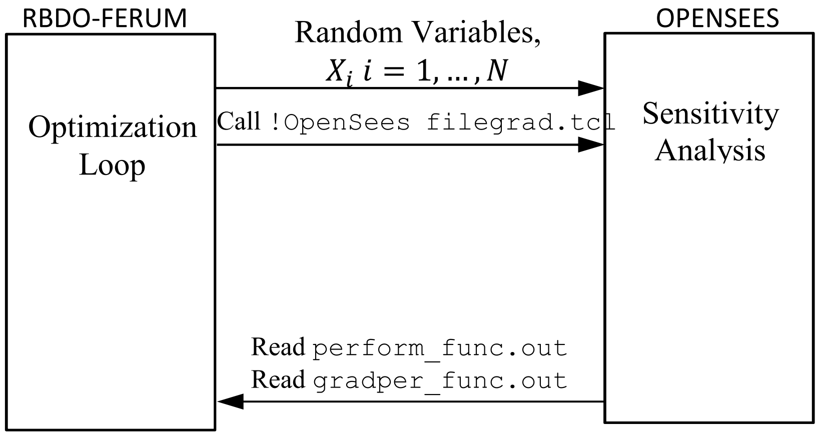

Figure 2. RBDO RIA-based double loop method

3. EXAMPLE

The probabilistic design problem consists in to compute de optima design variables for the structure represented in Figure 3. This steel frame is a modified version of the structural model described in the file `steelframe.tcl` available in the OpenSees Forum and analyzed in [3]. It is a three stories steel frame with three bays.

Members are grouped in three groups: Lateral Columns, Central Columns and Beams. All members assigned to a group have the same rectangular cross section, with width b=0.2m constant and height d. The height corresponding to each group is a random design variable. The Table contains random variables of the structure.
The problem can be formulated as:

$$
\begin{aligned}
&Min \quad V(\mathbf{d}, \boldsymbol{\mu}_{\mathbf{X}}, \boldsymbol{\mu}_{\mathbf{P}}) \\
&s.t. \quad P(g(\mathbf{d}, \mathbf{X}, \mathbf{P}) \leq 0) \leq P_f^t = \Phi(-\beta^t) \\
&\qquad \text{where} \quad \beta^t = 3.0 \\
&\qquad 0.1\,\text{m} \leq \mu_{d_j} \leq 0.5\text{m} \quad j = 1,2,3.
\end{aligned}
$$

To compute the design variables μ_{d_j} (mean values of d_j) that minimize the volume (weight) of the structure subject to a reliability constraint. $P(\delta_{x13}(\mathbf{d}, \mathbf{X}, \boldsymbol{P}) - 0.36m \leq 0) \leq \Phi(-\beta^t)$, where the target reliability index is β^t=3.

Geometric and material nonlinearities are considered in the analysis of the structure. These nonlinearities usually occur when the state of the structure is close to the limit state described in the performance function.

The probabilistic optimum obtained using PMA-based RBDO is: $d_1 = 0.18596\,m$; $d_2 = 0.18788\,m$; $d_3 = 0.18596\,m$. The number of Limit State Function evaluations is very high. However computational strategies can be carried out to reduce this quantity.

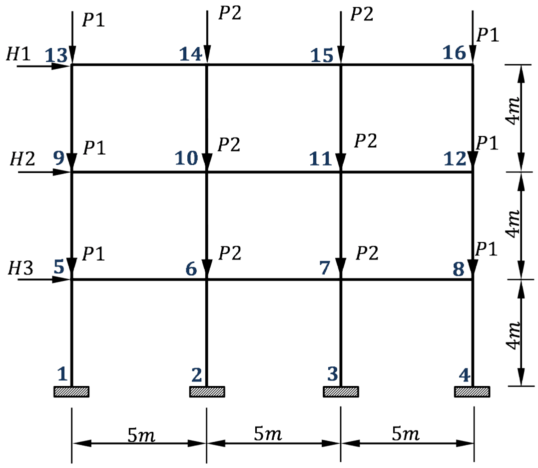

Figure 3. Steel Frame - 3 stories and 3- bays

Random Variable	Description	Dist.	Initial Mean	CoV	Design Variable
d_1	Height LC	N	0.4 m	0.02	μ_{d_1}
d_2	Height CC	N	0.4 m	0.02	μ_{d_2}
d_3	Height B	N	0.4 m	0.02	μ_{d_3}
E_1	Modulus LC	LN	200E+6 kPa	0.05	-
fy_1	Yield Stress LC	LN	300E+3 kPa	0.1	-
$Hkin_1$	Hard. Kin.LC	LN	4.0816E+6 kPa	0.1	-
E_2	Modulus CC	LN	200E+6 kPa	0.05	-
fy_2	Yield Stress CC	LN	300E+3 kPa	0.1	-
$Hkin_2$	Hard. Kin.CC	LN	4.0816E+6 kPa	0.1	-
E_3	Modulus V	LN	200E+6 kPa	0.05	-
fy_3	Yield Stress V	LN	300E+3 kPa	0.1	-
$Hkin_3$	Hard. Kin.V	LN	4.0816E+6 kPa	0.1	-
$H1$	Lateral Load	LN	400 kN	0.05	
$H2$	Lateral Load	LN	267 kN	0.05	
$H3$	Lateral Load	LN	133 kN	0.05	
$P1$	Vertical Load	LN	50 kN	0.05	
$P2$	Vertical Load	LN	100 kN	0.05	

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A Simplified Damage-following Model for Reinforced Concrete Columns

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SUMMARY

Nonlinear models are becoming a widely solution for seismic evaluation and design of new structures. The implementation of those models in engineering practice requires robust methodologies and guidelines about the consistent use of nonlinear analysis. Objective numerical models must be developed or, at least, the extent of the accuracy of each possible solution must be accounted for and its impact included within the safety verification framework. One particular issue that affects distributed inelasticity models is the consistency of the numerical model adopted. Not only is an analyst interested in global performance engineering demand parameters but also code-based procedures require that local response should be used in the assessment stage. Hence, it is important to reconcile local (Moment-curvature) and global (Force-drift) responses. The simplified damage-following models presented herein try to circumvent some issues associated with objectivity and user-dependent model selection. The considered models ensure correct capture of hardening behavior ranges and an objective response during softening. This is done by updating at each analysis step the inelastic zone length. A simple application is considered comparing experimental results from 1 RC column and regularized beam models available in OpenSees.

Key Words: *Damage, Regularization, Force-based, Reinforced Concrete, Model updating*

1 INTRODUCTION

The nonlinear cyclic analysis of RC members can be performed using different types of models which have been developed over the years and that require different levels of complexity in terms of modelling, analysis and interpretation of the results. From a practical point of view, frame models constitute an interesting option since, in Earthquake Engineering, the number of analyses that are required to assess the behaviour of a structure usually demands for a simple approach.

Nevertheless, the level of accuracy needs to be considered, since the simplicity of the model may involve inherent simplifications and omissions that can have a strong impact in the seismic demand results. The present study presents a simplified alternative method that can be used in OpenSees to evaluate the reliability of some of the current consistent methods involving static formulations (without adaptive properties) towards the implementation of modelling uncertainties in current performance-based methods.

2 SIMPLIFIED ADAPTIVE FORCE-BASED ELEMENT

Using a *low order* integration scheme available in OpenSees (Scott, 2011), an adaptive force-based beam column element based on Gauss-Lobatto integration was constituted, relying on the classical configuration of the method while hardening sectional response is observed and shifting towards a regularized approach when softening effects arise. This numerical method allows for the explicit specification of the position of all integration points (IPs) along with the correspondent integration weights.

In the proposed method, the extreme IPs (i and j) assume, when softening is verified, an integration weight which is equivalent to the characteristic length (L_p) of the beam column element. Several mathematical models can be assumed to compute the characteristic length, with higher or lower levels of complexity, including static data (i.e. constant modelling parameters) or response-based data (i.e. from the output of the analysis) to compute their values. Almeida *et al*. (2012a) defined that once softening is observed in a structural element, the IP weights must be recomputed under the assumption that the extreme integration weights are equal to λ_p. The value of λ_p can be calculated according to a given integration interval [a; b] using Eq.1.

$$\lambda_p = (b-a)\cdot\frac{L_p}{L} \tag{1}$$

When softening is identified in an integration point at the extreme position i, the correspondent integration point must reflect the expected characteristic length of the section. Hence, in this case the IP1 weight must be set in order to match the normalized characteristic length $w_1^* = \lambda_{pi} = 2\cdot L_{pi}/L$, since it is assumed that the domain [1; 1] corresponds to yielding (b-a)=2. Using interpolatory quadrature and taking the Vandermonde matrix, the additional integration weights can be computed according to Eq. (2). A similar strategy can be defined for the analysis of the additional cases, comprising the case when the IP at location j presents softening behaviour or representing the situation in which both extreme IPs enter in a softening range

$$\begin{bmatrix} w_2^* \\ w_3^* \\ w_4^* \\ w_5^* \\ w_6^* \\ w_7^* \end{bmatrix} = \begin{bmatrix} 0.391640 - 2.411088\cdot\lambda_{pi} \\ 3.011088\cdot\lambda_{pi} + 0.2883602 \\ 0.640000 - 3.200000\cdot\lambda_{pi} \\ 3.011088\cdot\lambda_{pi} + 0.2883602 \\ 0.3916398 - 2.411088\cdot\lambda_{pi} \\ \lambda_{pi} \end{bmatrix} \tag{2}$$

Using the routine *updateparameter* available in OpenSees, the initially defined integration weights (Gauss-Lobatto with 7 IPs) are updated to the new values and these new values are used in the next analysis step. The curvatures and bending moments recorded in the updating stage are taken also as the reference values to assess the softening condition in the next step.

An alternative method can also be considered using inelastic zone force-based elements has the one proposed by Scott and Fenves (2006). In this context, the regularized strategy formulated can be used to update the parameter Lp_1 and Lp_2 and the integration weights can be consistently obtained directly using the beam formulation available.

3 DAMAGE FOLLOWING CRITERIA FOR ADAPTIVE-MODEL

The adaptive formulation defined in the previous section requires that a criterion must be selected to update the characteristic length. Three alternatives can be considered in order to consistently represent the spread of the inelastic zone length in the beam columns.

a) A first method can be seen has an extension of classical models, and involves a single updating transformation of the initial hardening scheme suitable model to an inelastic zone model when a trigger condition (curvature or moment softening condition as proposed by Almeida *et al*. (2012a).
b) A second approach involves the use of an adaptive characteristic length at both sides of the structural element considering a linear development of the moment distribution along the height of the column. Using a reference moment (for instance the yielding moment or the concrete cover spalling moment) to compute the characteristic length, it is possible to identify the extent of the damaged zone and recalculate the integration weights in order to account for actual size of the inelastic zone. This can be done either using the full computation defined before or the simple strategy updating the hinge length in the Scott and Fenves (2006) model available at OpenSees.
c) A last approach can be seen as a fully automatic version of the adaptive model and involves the longitudinal steel strain profile in compression along the length of the member and using piecewise cubic interpolation functions. This approach has been proposed by Almeida *et al*. (2012b) and uses the constructed profile, which can benefit from the fact that 7IPs can be used to ensure hardening consistent responses, to compute the characteristic length corresponding to the extent of the reinforcing bar length that exceeds the yielding strain. In the present study, due to the update strategy assumed (i.e. updating at the end of each analysis step), an initial trigger condition was established which corresponds to the yielding curvature at the extreme section being achieved and the updating strategy was induced at every step at which an higher value than the previously recorded curvature is verified. An application of this strategy was studied in the present article. The main objective was to observe if a strategy involving instantaneous recorders for the response (available at OpenSees) in combination with the *parameter/updateparameter* commands would allow for the use of a simplified version of a damage-following beam-column element.

4 APPLICATION

The experimental results of the specimen S24-5T tested by Bae (2005) is presented herein for the evaluation of the proposed adaptive strategy. The column is 3.048 m high and has a sectional configuration according to figure 4, with 0.609 x 0.609 m^2. Details of the column and materials can be found in Bae (2005). The analysis of the results obtained for the strain based adaptive scheme composed by 7 IPs showed that a considerable level of accuracy was obtained with the current model. An approximate value of the experimental damage (0.28m) length was obtained

(Figure 1b). Figure 2 shows the consistent results also obtained for the global and the local response at the base section of the column even at large softening stages.

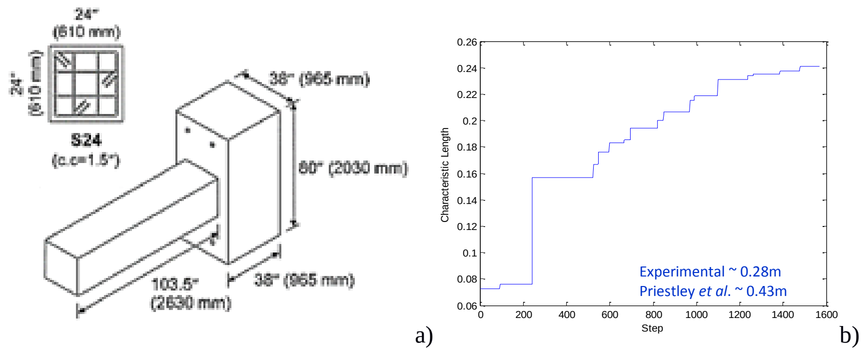

Figure 1. Case study: column S24-5 from Bae (2005).

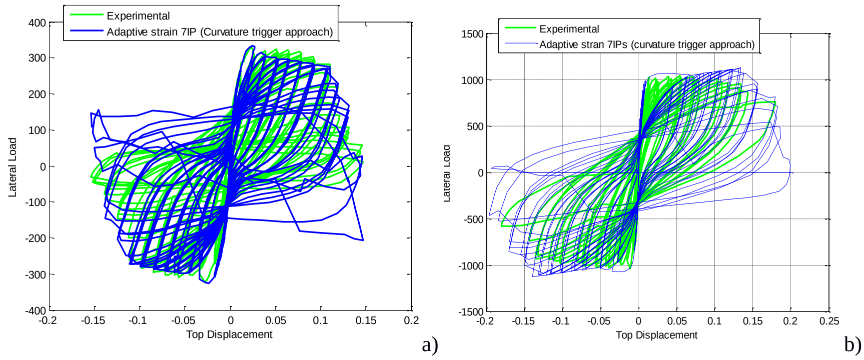

Figure 2. Comparison of the obtained results with the experimental local and global column responses

5 CONCLUSIONS

An application of a damage-following adaptive force based element has been presented and its performance against one experimental case with moderate axial loads was also presented. Simplified methods have been formulated, allowing for the use of adaptive beam formulations in OpenSees. Promising results were obtained, but further studies are necessary to assess the robustness of the implemented approach. Its use as a tool to evaluate the probabilistic comparison and reliability of more simple methods can be accepted, evaluating the impact of the modelling strategies selected by the analyst.

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Multi modal response spectrum analysis implemented in OpenSEES

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SUMMARY

In seismic design, the most commonly used analysis method is the multi-mode response spectrum analysis (MMRSA) due to its relative simplicity. OpenSEES is a powerful tool for seismic analysis of structures, but lacks the in-built option for MMRSA, while this linear-dynamic analysis method can be advantageous if quick results are needed, e.g. in case of a parametric study. MMRSA is implemented in TCL language by the authors, the application and boundaries of the MMRSA procedure is presented through some case studies.

Key Words: *earthquake engineering, response spectrum analysis.*

1 INTRODUCTION

OpenSEES is a powerful open source platform for seismic analysis, yet multi modal response spectrum analysis (MMRSA) which is the most commonly used earthquake analysis method among engineers is not integrated in the software. Although, MMRSA tends to overestimate the seismic responses, and can be used only for linear analysis, it can provide fast results and generally an upper bound for the developing internal forces. The shorter computational time of MMRSA can be advantageous if parametric study should be carried out. Typically, sensitivity analysis or the analysis of structural archetypes with different geometric layouts to identify the most critical input parameters or critical structural components regarding seismic actions are considered such parametric study. After this identification, more sophisticated analysis methods can be applied based on the results.

2 MULTI MODAL RESPONSE SPECTRUM ANALYSIS

Theoretical background

Response spectrum analysis is a well-known analysis method: the main idea and the implementation of the theory can be found in the literature (see for instance [1]). However, it is necessary to present the basic equations to understand the application of the procedure written in TCL language.

MMRSA is based on the modal analysis of the structure, thus as a first step, the eigenvectors describing the modal shape and the eigenvalues representing the frequency of a specific mode should be determined. The importance of a mode can be characterized by its modal mass (m_i^*), while the load vector ($\boldsymbol{p}_i$) for each mode can be calculated:

$$m_i^* = \frac{\left(\boldsymbol{\Phi}_i^T \boldsymbol{m}\, \boldsymbol{\iota}\right)^2}{\boldsymbol{\Phi}_i^T \boldsymbol{m}\, \boldsymbol{\Phi}_i} \quad , \qquad \boldsymbol{p}_i = \boldsymbol{m}\, \boldsymbol{\Phi}_i \frac{\boldsymbol{\Phi}_i^T \boldsymbol{m}\, \boldsymbol{\iota}}{\boldsymbol{\Phi}_i^T \boldsymbol{m}\, \boldsymbol{\Phi}_i} S_d(T_i) \quad , \tag{1}$$

where m_i^* is the modal mass, $\boldsymbol{\Phi}_i$ is the eigenvector, $\boldsymbol{p}_i$ is the load vector, T_i is the natural period and $S_d(T_i)$ is the design acceleration spectrum value for the *i*-th mode and $\boldsymbol{m}$ is the mass matrix. Once the quasi-static loading is obtained for the necessary number of modal modes (typically, the sum of the modal masses should reach 90% of the total mass in each direction), static analysis can be carried out to determine seismic responses for each mode. The response parameters corresponding to given modes (e.g. internal forces, displacements, etc.) can be then combined with one of the following methods (ABSSUM, SRSS or CQC*)*:

$$E_{ABSSUM} = \sum |E_i| \ , \qquad E_{SRSS} = \sqrt{\sum (E_i)^2} \ , \qquad E_{CQC} = \sqrt{\sum_{\boldsymbol{i}} \sum_{\boldsymbol{j}} E_i\, \rho_{ij}\, E_j} \quad , \tag{2}$$

where E_i is the seismic response for the *i*-th modal mode, ρ_{ij} is the correlation coefficient due to Kiureghian [1], E_{ABSSUM}, E_{SRSS} and E_{CQC} are the summarized results. The resulting parameters shall be realized in all relevant directions of seismic excitation, and the different direction results shall be combined according to the seismic provision (e.g. 30% rule, [1]).

Response spectrum analysis procedure in TCL

The response spectrum analysis (RSA) procedure based on the above presented theoretical background is written in TCL language, but it can be implemented in the source code directly if necessary. The main purpose of the procedure is to be able to carry out three dimensional analysis of bridges. The procedure name and the input parameters are:

proc ***RSA*** *{a_g soiltype eqtype ξ q q_d sumtype elelist nodelist},*

where a_g is the peak ground acceleration [m/s^2], *soiltype* (A, B, C, D or E) and *eqtype* (1 or 2) define the shape of the design response spectrum according to Eurocode 8-1 [2], ξ is the damping ratio, q is the behavior factor, q_d is needed to calculate displacements (generally its value is the same as q except for short periods), finally *sumtype* (ABSSUM, SRSS, CQC) defines the combination method according to *Eq.2*. Efficiency of the combination increases if only necessary

responses of desired elements and nodes are summarized and stored. Input variables *elelist* and *nodelist* contain these elements and nodes in TCL list form, if not defined, summarized responses of ALL elements and nodes are calculated.
First, modal analysis is carried out, modal shapes and frequencies are determined with the *eigen* command. The authors emphasize that the mass matrix is constructed by the *nodeMass* command, and thus in the numerical model the structural mass shall be modeled as lumped mass in the nodes. Modal masses and number of required modes are calculated according to *Eq.1* and [2], respectively. For each modal mode, the load vector is constructed and static analysis is carried out. Only displacements of nodes and internal forces of elements listed in *nodelist* and *elelist* are stored. The procedure cannot recognize the element type of the listed elements, the *[eleResponse $element_number forces]* is used to get element results. Thus in current state, only element results with output type *forces* (e.g. elasticBeamColumn) can be queried. The stored results then can be combined with one of the methods presented in *Eq.2* and defined with *sumtype* as input parameter. The combined displacements and reaction forces are set as global variables and can be used later during the analysis. The form of the output variables can be seen in *Fig.1*.

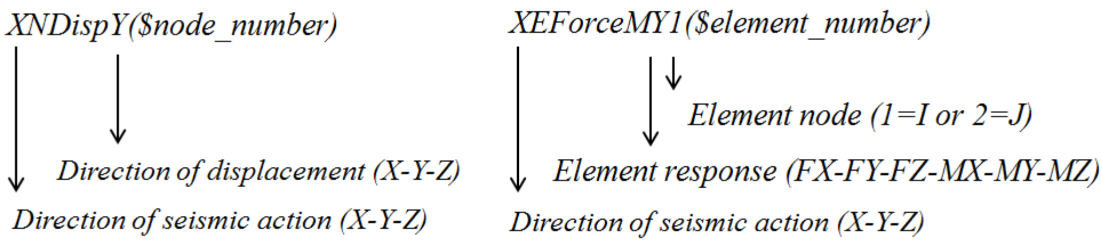

Fig.1. Output variables for the RSA procedure in TCL.

Only translational displacements (*NDispX, Y* and *Z*) are calculated in the global coordinate system (GCS), while element internal forces (*EForceFX1, FY1, FZ1, MX1, MY1* and *MZ1*) are calculated both endnodes (*I* and *J*) of the element in the GCS. With respect to the seismic action, there are 3·3=9 and 3·2·6=36 global list variables for displacements and element internal forces, respectively.

3 EXAMPLES OF APPLICATION

The RSA procedure is applied in many studies by the authors. In [3], the seismic assessment of an existing Hungarian highway bridge is presented. The investigated large-span bridge consists of continuous steel box-section superstructure supported by reinforced concrete piers. A beam-element numerical model is developed in OpenSEES, which incorporates non-linear characteristics of the critical details. Pushover and incremental dynamic analysis is applied to determine the fragility curves reflecting the probability of various damage occurrences. Here, the linear RSA results are used control and give an upper-bound for higher level analysis results.
In [4], three continuous girder bridges are studied with seismic isolators. The mechanical behavior of isolation devices is usually non-linear, but as per Eurocode 8 [2], they can be modeled with equivalent linear stiffness and effective damping. This results in an iterative analysis method programmed within the RSA TCL procedure. RSA and non-linear time-history analysis results are compared and then the applicability and validity of the effective stiffness method is presented in the paper.

In Hungary, the number of prestressed multi-girder bridges counts for nearly half of the of higher level road bridges. In [5], MMRSA is used for parametric studies to figure out how vulnerable these bridges are to seismic actions. Main variable parameters are: number of supports (2-5), span of single bay (4-24 m) and pier height (2-14 m). The results indicate that the superstructure, the abutments and the backfill soil are normally satisfactory with respect to strength. However, the abutment-superstructure, pier-superstructure joints as well as the pier and foundation are found critical at longer bridges. The quick RSA results provide solid base both for detailed seismic assessment and retrofit planning as well as for development of design concepts of newly built structures in moderate seismicity zones.

4 CONCLUDING REMARKS

The authors developed a general OpenSEES modul for response spectrum analysis of structures. Benefits of RSA is shown through three examples. The results obtained from higher level modelling and analysis methods can be controlled [3]. Besides, non-linear elements can be handled with effective stiffness and iteration methods [4]. Shorter computational time can be advantageous if parametric study should be carried out [5].
The RSA procedure written in TCL language should be implemented in the source code, so that the mass matrix can be constructed even with discontinuous, unlumped masses. During the combination of the responses, the element type can be recognized and specific responses can be queried, stored.

ACKNOWLEDGEMENT

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Deteriorating hysteresis model for cold-formed steel shear wall panel based on physical and mechanical characteristics

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SUMMARY

Cold-formed steel shear wall panel (CFS-SWP) is the main lateral load resisting system in the CFS wall panel building system. It is made of CFS framing members attached to steel or wood sheathing using screw connections. The complex analysis and design procedures related to the significant number of thin components locally unstable, and having several failure mechanisms, require a special seismic behaviour investigation. Full-scale testing is a good tool to be employed to understand their performance under significant seismic events, but it is very tedious and costly. Hence, a model which is able to simulate as accurately as possible the shear wall panel's response, is of a primary importance. In this paper, a smooth hysteresis model that takes into account strength and stiffness deterioration, as well as pinching effect has been developed. The model parameters are related to the physical and mechanical characteristics of the CFS SWP; the latter has been implemented in OpenSees program, as a user-defined uniaxial material named CFSSWP. The accuracy of the proposed model is validated against the experimental test results obtained from literature where a good agreement has been achieved.

Key Words: *cold-formed steel, shear wall panel, hysteresis model, OpenSees, non-linear analysis.*

1 INTRODUCTION

In recent decades, the cold-formed steel (CFS) sections are increasingly used in a low to medium rise buildings as primary elements, even in seismic prone regions. In which, the shear wall panel (SWP) is the main lateral load resisting system; it is made of CFS C-shaped framing members (studs and tracks) attached to steel or wood sheathing using screw connections. Under repeated cyclic deformation, the formed hysteresis loops are characterised by severe strength and stiffness deteriorations as well as a pinching effect. These phenomenons which affect most the post-elastic behaviour must be taken into account in the dynamic nonlinear analyses. The basic requirement to perform such an analysis is the availability of an accurate constitutive model capable of simulating as accurately as possible the SWP response when subjected to a quasi-static or dynamic lateral loading. In this paper, a smooth hysteresis model for CFS SWP based on the model developed by Lowes LN and Altoontash A (2003) [1] that takes into account strength and stiffness deterioration with pinching effect, has been illustrated. The outstanding advantage of this model is that all its parameters are directly related to the physical and mechanical characteristics of the SWP. The accuracy of the proposed model is validated using experimental results obtained from literature.

2 HYSTERESIS MODEL

The uniaxial hysteresis model of CFS SWP consists of three parts: backbone curves of the hysteresis loops (states 1 and 2), hysteresis criteria (unloading-reloading path: states 3 and 4) (Fig.2) and deterioration criteria. The following paragraphs will respectively introduce the expressions of the three parts.

Maximum lateral shear strength and the associated displacement are assessed using two analytical methods for wood sheathed and steel sheets sheathed CFS SWP proposed by, respectively, Xu L and Martinez J (2007) [2], and Yanari N and Yu C (2013) [3] which take into account a wide range of factors that affect the behaviour and strength of a CFS SWP, namely: material properties, thickness and geometry of sheathing and framing, spacing of studs, construction details such as size and spacing of sheathing-to-framing connections.

A curved envelope is adjusted by applying the B-Spline algorithm; this achieves the curvature for the states 1 and 2 of the hysteresis model (Fig.2). As can be seen in Fig.1, a good agreement between the envelope curves of a SWP developed analytically and the one derived from experimental monotonic tests for specimen n° 25 tested by Branston et al., (2006) [4] and specimen 1Mb tested by Balh (2010) [5].

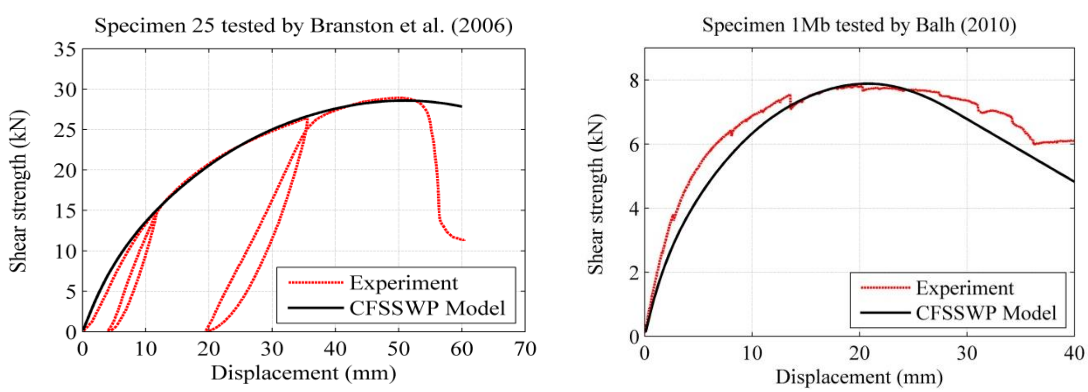

Figure 1. Comparison between analytical and experimental monotonic curves

In addition to the envelope curve, the proposed hysteresis model requires the introduction of parameters that define the strength and stiffness deterioration, as well as the pinching effect under cyclic loading. Compared to the monotonic test result, the hysteresis response of CFS SWP exhibits strength deterioration; even if the displacement associated to peak strength has not been reached yet. The stiffness deterioration of the proposed model is positively related to strength degraded degree, and is defined in a same way as the strength deterioration.

On the basis of information deduced from the CFS SWP experimental database, empirical relationships between the deterioration parameters and the physical and mechanical characteristics of the CFS SWP have been defined. The hysteretic criteria and the deterioration criteria are shown in Fig.2. The curvature for the states 3 and 4 is obtained with the monotone cubic spline algorithm as the generated curve always passes through the 4 points in phases 3 and 4 while ensuring the monotony of the curve.

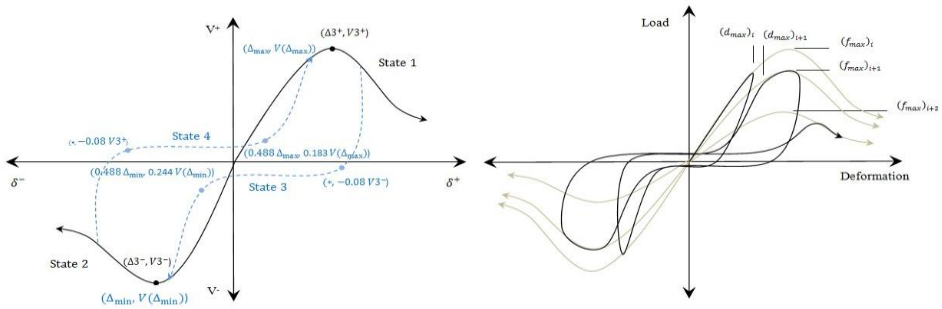

Figure 2. Four states of the proposed hysteresis model and impact of cyclic deterioration on load-deformation response

3 NUMERICAL SIMULATION

In order to account for the overall lateral stiffness and strength of the SWP, an equivalent simple non-linear zeroLength element with CFSSWP model connected to rigid Truss elements which transmit the force to the end elements (chord studs) that resist to axial tension and compression stresses (Fig.3). This modeling tip lead to a considerable reduction in terms of element number constituting the CFS SWP. The boundary members form a mechanism and lateral strength and stiffness are derived directly from zeroLength element. The CFS SWP details, as well as the schematic representation of finite element (FE) model are illustrated in Fig.3.

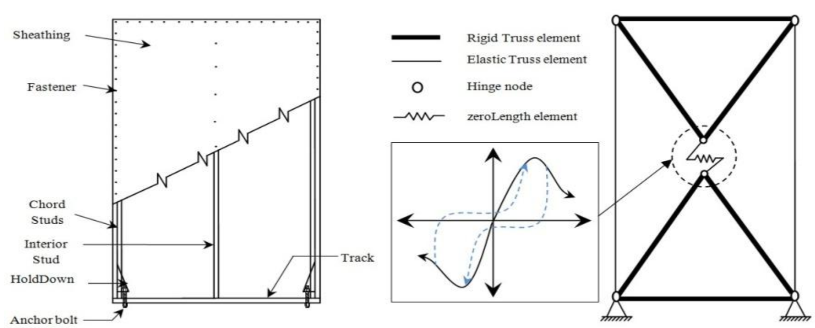

Figure 3. CFS SWP details and equivalent OpenSees FE model

In an attempt to check the accuracy of the proposed CFSSWP model, quasi-static non-linear analyses of CFS SWPs have been carried out using OpenSees software. Specimens n° 32 tested by Branston et al. (2006) [4], and specimen 3C-a tested by Balh (2010) [5] have been selected from literature in order to validate the proposed approach. More details about the experimental program and results, as well as the modeling process are reported in [4-6].

The shear strength-lateral displacement hysteresis loops of the aforementioned specimens from tests are plotted together with numerical model results in Fig.4.

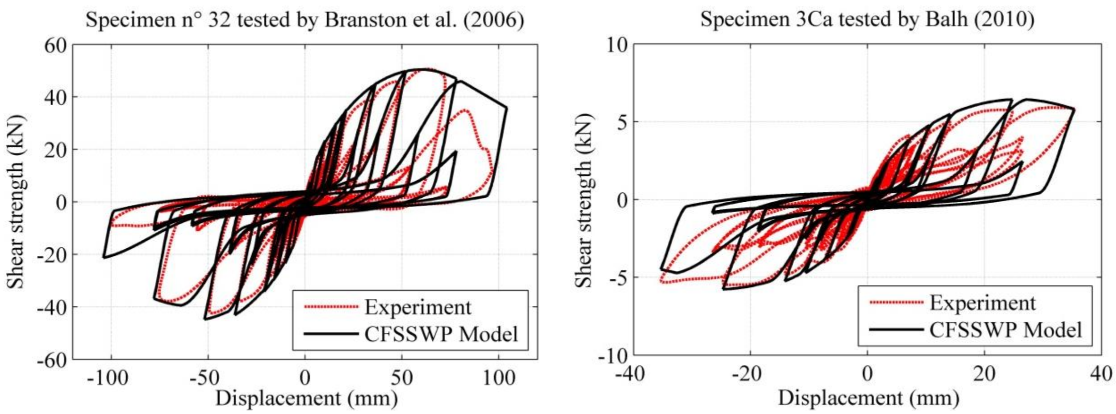

Figure 4. Comparison between wood sheathed and steel sheathed CFS SWP experimental and numerical results

4 CONCLUSIONS

This paper presents an analytical approach to predict hysteresis behaviour of a wood/steel sheathed CFS SWP with deterioration criteria based on its physical and mechanical characteristics. The model has been implemented into the finite element software OpenSees as a user-defined uniaxial material using a DLL written in C++ programming language. A non-linear analysis is carried out using a macro-element panel which incorporates the equivalent constitutive model. The efficiency of this modeling tip and the accuracy of the proposed hysteresis model have been proved by a good agreement between the experimental and the numerical results in terms of shear strength-lateral displacement relationship where the fundamental response characteristics of the CFS SWP such as: strength and stiffness deterioration, as well as the pinching effect are reasonably well simulated. The difference between the positive and negative loops' performance of the cyclic response has been also well captured using CFSSWP model.

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Numerical Modelling of masonry infill walls participation in the seismic behaviour of RC buildings

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SUMMARY

Recent earthquakes show that masonry infill walls should be taken in account during the design process, because these types of non-structural elements increase the in-plane stiffness of the structure and consequently the natural period of the structure. The out-of-plane behaviour of masonry infill walls is observed as serious problem putting the peoples' security in risk. In this study, an improved numerical model for simulating the masonry infill walls behaviour when submitted to seismic actions is proposed. This model is adapted from the model proposed by Rodrigues to use in OpenSees. The main results of the in-plane calibration analyses obtained with one experimental test, in LNEC, are presented and discussed. Two reinforced concrete (RC) regular building was observed and submitted to several ground motions, considering and not the infills' out-of-plane behaviour. The results are presented in terms of damaged, inter-storey drift, capacity curves and energy dissipated.

Key Words: *masonry infill walls, in-plane behaviour, regular RC buildings, vulnerability*

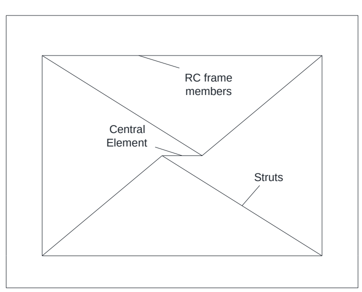
1 INTRODUCTION

The presence of masonry infill walls in reinforced concrete (RC) buildings is very common in south European construction. However, in the design of new buildings and in the assessment of existing ones, infills are usually considered non-structural elements and their influence on the structural response is neglected. The main objective of this work was to use a modelling strategy similar to the model developed by Rodrigues [1] to represent the masonry infills behaviour in OpenSees using the elements and the uniaxial non-linear material models available. The in-plane model calibration was performed based on experimental test performed by Pires [2].

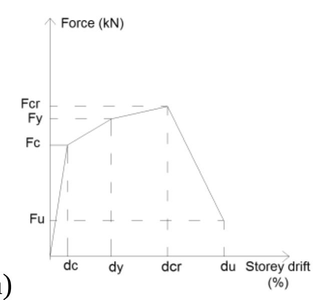
2 PROPOSED MASONRY INFILL WALL MODELLING PROCEDURE

The macro-model used in OpenSees is based on the Rodrigues et al. [1] proposal which is an improvement on the commonly used equivalent bi-diagonal-strut model. Therefore, the proposed model tends to accurately represent global response and energy dissipation during structural response.

In this model [1], each infill panel is defined by considering four support strut-elements, with rigid in plane behaviour, and a central element, where the in-plane non-linear hysteretic behaviour is concentrated Figure 1a. The forces developed in the central element are purely of tensile or compressive nature when submitted to in-plane solicitations. In the initial idealization the non-linear monotonic behaviour was characterized by a multi-linear curve, defined by eight parameters (Figure 1b), representing: a) cracking (cracking force F_c and Cracking d_c); b) yielding (yielding force F_y and yielding displacement d_y); c) maximum strength, corresponding to the beginning of crushing (F_{cr} and corresponding displacement d_{cr}); d) residual strength (F_u) and corresponding displacement (d_u). The proposed macro-model for masonry infill walls was modelled in OpenSees by the association of the available OpenSees materials, sections and elements commands. The infill model is composed by 4 Elastic BeamColumns for the diagonal elements and by one nonlinear beamcolumn element for the central element. For the central element, where the non-linear behaviour is concentrated, the Pinching4 material model was used to represent the non-linear hysteretic behaviour and the envelope curve was calibrated with experimental characteristics of the infill's. This uniaxial material is used to construct a uniaxial behaviour that represents a 'pinched' load-deformation response and exhibits degradation under cyclic loading. Cyclic degradation of strength and stiffness occurs in three ways: unloading stiffness degradation, reloading stiffness degradation, strength degradation, and this parameters were calibrated based on the available results.



a)



b)

Figure 1 – a) Macro-model b) hysteretic material behaviour for the simulation of a masonry infill panel (adapted by Rodrigues).

3 CALIBRATION OF THE PROPOSED MODEL

The proposed macro-model for masonry infill walls was tested and calibrated with the results of a cyclic in-plane test performed by Pires [2]. The infilled reinforced concrete is a single-bay, scaled 2:3. The geometrical characteristics of the frame, the cross sections dimensions and the reinforcement detailing of the columns and beam are presented in Figure 2. The axial load was applied to the top of the columns, to simulate the dead load, and imposed cyclic horizontal displacements were applied. The RC frame was modelled with beam with hinges elements and calibrated with the results of materials properties. The results of the numerical model with masonry infill walls was calibrated with the results of the experimental results.

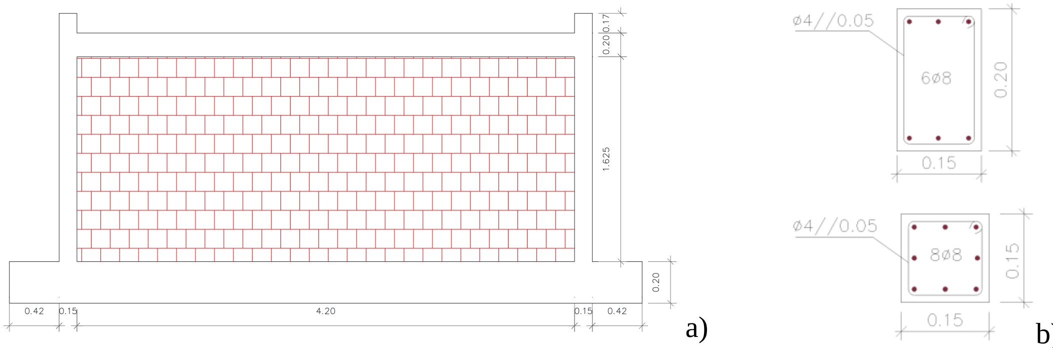


Figure 2 – Single-story single-bay infilled masonry RC frame: a) frame geometry b) Cross sections dimensions and detailing of RC elements.

The results obtained with the numerical model are in good agreement with the experimental response in terms of shear-drift (Figure 3a.) and energy dissipation (Figure 3b), demonstrating the ability of the proposed model to simulate the global hysteretic response of infilled frames.

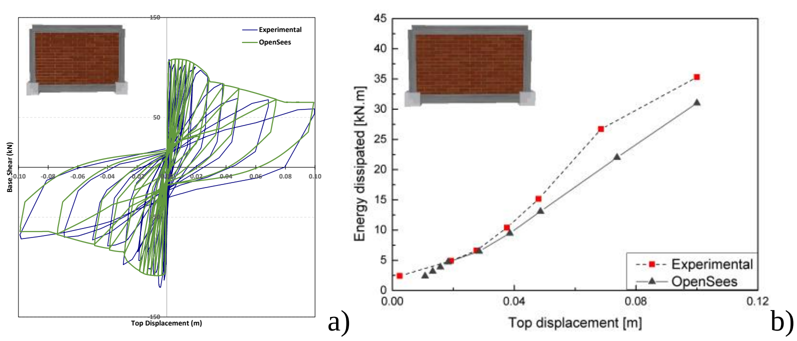


Figure 3 – a) Shear – Top displacement and b) Energy dissipated results of experimental test and the numerical calibrated model.

In order to compare the best fit monotonic curve used to simulate the behaviour of the test, it was adopted for the reference parameters of the central element the proposed values by Zarnic and Gostic [3] and later modified by Dolsek and Fajfar [4] which are based on recommendations from past researchers, experimental tests and is in accordance with the FEMA 356 [5] recommendations. In Figure 4a is illustrated the base-shear top displacement envelopes and it can be observed that the general model results are 5-10% higher in terms of base shear for the same top displacement values. The energy dissipated (Figure 4b) was determined and once again the general model was 10-20% higher than the experimental response, which is acceptable because it's a simplified model, modelling the entire infill panel and its surrounding frame elements.

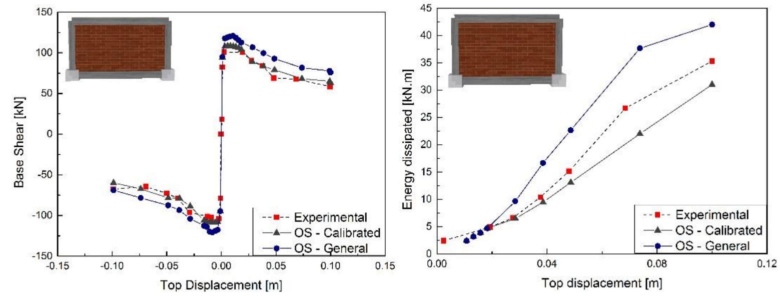


Figure 4 –Comparison of the numerical models results: a) base-shear – top displacement envelopes b) Energy dissipation.

3 CONCLUSIONS AND FUTURE WORK

This paper describes a proposed simplified macro-model for the representation of the hysteretic behaviour of infill masonry panels, and its implementation on OpenSees. The masonry model proposed is adapted from the Rodrigues proposal model, considering the interaction of masonry panel behaviour in both directions, i.e. damage in the panel in one direction affects its behaviour in the other direction. The proposed infill masonry model was calibrated with the results of one experimental test and the results were in a good agreement with the experimental response. The limitation of the proposed model is that doesn't account for the short column effects. For infilled frames where the short column can be induced, multiple strut model strategy should be adopted.

Two, 3D, RC buildings models (2, 4 and 6 storeys) were studied to observe the influence of the presence of the masonry infill walls in regular structures. An incremental dynamic analysis (IDA) was performed and the results and discussion are presented in terms of damage ratio in the RC elements and in the infill's.

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Application of OPENSEES for Seismic Progressive Collapse Evaluation of Asymmetric RC Moment Resisting Buildings

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ABSTRACT

Margin of safety against potential of progressive collapse is an important property of a structural system. Often eccentricity in plan of a building causes concentration of damage, thus adversely affects progressive collapse safety margin of that building. In this paper some modeling issues in application of OPENSEES for seismic progressive collapse assessment of reinforced concrete moment resisting buildings with eccentricity in plan are studied. The building models are 0%, 5%, 15% and 25% mass eccentric 6-story reinforced concrete ordinary moment frame buildings. The distribution of damage and spread of collapse is investigated using nonlinear time history analyses. An important modeling issue is selecting the type of nonlinear element capable of reliably identifying collapse of a building. Results show concentrated plasticity approach is a more appropriate tool for such studies compare with other available techniques such as fiber modeling. Using such models show that increase in potential of progressive collapse at both flexible and stiff edges of buildings with increase in level of asymmetry in buildings. It is also demonstrated that "drift", as a more easily available global response parameter of frame, is a good measure for a much difficult to calculate local response parameter of "number of post collapse plastic hinges". Among other important issues in progressive collapse behavior of a building is the type and location of the damaged elements. This paper also deals with identifying the effects of distribution of collapse from the first element to the entire of the structure due to earthquake loads. Here, 3D collapse propagations in symmetric and asymmetric reinforced concrete buildings are compared using nonlinear time history analyses. The variables of such analyses are earthquake load intensity and level of mass eccentricity. Results show that collapse distribution is dependent on the degree of asymmetry in building. Some patterns to predict progressive collapse scenarios in similar symmetric and asymmetric buildings are also determined. One main pattern shows that propagation of collapse is horizontal through the stories not vertical through the height of the buildings. Spread of the collapse is independent of the earthquake records. According to the results, probability of collapse initiation from the columns increases when mass eccentricity increases. Generally, collapse in asymmetric buildings initiates mainly on the flexible edge of the building.

Key words: Asymmetric buildings, Progressive collapse, OPENSEES, Mass eccentricity, Reinforced concrete ordinary moment resisting frame building, Nonlinear time history analysis.

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Numerical simulation of the seismic behavior of RC bridge populations for defining optimal intensity measures

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SUMMARY

Existing reinforced concrete (RC) bridges might be vulnerable to seismic events hence characterizing their vulnerability is a particularly relevant aspect when dealing with seismic loss assessment of e.g. the road network of a certain region. Traditionally, structural (or physical) vulnerability is defined by means of fragility curves, which are in turn defined as function of an intensity measure (IM) characterizing the intensity of the seismic action. Currently one can find abundant available literature on the investigation of the correlation of estimated seismic response with sets of both scalar and vector IMs however such studies have been mostly applied to buildings and from a single-structure perspective. The present study intends to overcome both limitations by revisiting the intensity measures topic and by comparing the performance of traditional and newly proposed IMs when applied to bridge portfolios, with a view to application in risk assessment. To accomplish this, a significant number of nonlinear analyses is necessary, due to the considered large number of randomly simulated structural configurations (bridge population) and intensity measures (23 in total). A global script, using OpenSees and Matlab was coded to automatically run a large number of time-history analyses, as well as statistically process the results. The generated script covered the initial statistical characterization of the main bridges properties of the randomly generated population and the generation of the input files for performing nonlinear time history analyses, assured by OpenSees. The post-processing procedure was the last stage, through which correlation between selected engineering demand parameters (EDPs) and tested IMs were established, leading to the identification of the most promising intensity measures for vulnerability assessment of bridge populations.

Key Words: *numerical simulation, OpenSees RC bridge modelling, seismic behavior, intensity measures, probabilistic seismic demand, Latin Hypercube Sampling.*

1 INTRODUCTION

During the last decades, research has been mainly focused on the improvement of procedures for the assessment of individual structures however if one wants to assess the impact (in terms of damage and losses) of an extreme event, such as an earthquake, over a vast area, the consideration of large populations of structures is necessary. Indeed, in the recent years, much research attention has been paid to the analysis of a larger number of structural configurations, as part of systems. These approaches are particularly appropriate when bridge structures are considered within road networks. Such structures play a key role in the economic, social and emergency management context therefore for a proper evaluation of the seismic risk they are posed at, the inclusion of uncertainties in both seismic hazard and structural response is crucial. One of the tools that couples Probabilistic Seismic Hazard Analysis (PSHA) and nonlinear structural analyses is the so-called Probabilistic Demand Model (PSDM) [1], which is based on the probability of reaching a certain limit state (defined in terms of a specific EDP), conditioned to a given intensity measure. Defining a proper intensity measure is thus essential to reduce the uncertainty associated with PSDMs while catering for more accurate fragility estimates and it represents the main goal of the present study, which compares both scalar and vector-IMs when using an extensive number of different bridges. Each bridge, generated with Latin Hypercube based sampling [2], was subjected to a set of thirty ground motion records, scaled according to the Conditional Spectrum method [3], for seven different intensity levels. The accomplishment of such large number of analysis was possible in reasonable time due to the implementation of automatic interaction between Matlab [4] and OpenSees [5] for selecting input records, performing nonlinear dynamic analyses and post-processing the results of around 21’000 nonlinear dynamic analyses. Structural response was characterized by means of five EDPs, expressed in terms of displacements or ductility. The quality of the correlation between the response estimates and the traditional or innovative intensity measures was assessed in terms of efficiency, practicality and proficiency [6] through regression analyses in the transformed space. Pearson Product Moment Correlation Coefficients were then extracted for investigating the robustness of the assumed predictive model.

2 PARAMETRIC STUDY

The selected case study is made up of a hundred reinforced concrete bridges randomly generated using the Latin Hypercube [2] sampling method, procedure chosen for the main advantage of reducing computational time demand and validated in previous research for the estimate of nonlinear response of bridges [7]. All bridges represent typical existing Italian bridges, as part of the road network located in the Italian Central Region of Molise, whose seismic hazard was characterized using OpenQuake platform [8], on the basis of the seismic source zones of the Italian zonation [9]. Main geometrical and material properties were thoroughly characterized by the means of statistical distributions, fitted according to Chi-Square or Kolmogrov-Smirnov goodness-of-fit test satisfaction. A main Matlab script controlled this initial stage, fundamental for the definition of the bridge configurations and the generation of the ‘.tcl’ input files of each bridge model, enabling the settlement of an automatized procedure. OpenSees is then called in, as the nonlinear structural analysis tool, for running eigenvalue analysis in order to properly select the set of ground motion records corresponding to the fundamental transversal period of vibration, used as input for the nonlinear dynamic analyses. Each bridge model was composed by four main classes of components: the piers, each one modelled through a single force-based element and discretized in fibres at sectional level for capturing their nonlinear behavior; the deck, modelled as a continuous elastic element, located at its centre of mass and connected to the piers through rigid links; shear keys, defined as zero length elements and equal-degree-of-freedom connections and the abutments, simulated as high stiffness springs with bilinear response. Equivalent SDOF maximum displacement, maximum mean top displacement, maximum displacement, maximum column ductility and maximum displacement of the shortest pier were extracted from nonlinear dynamic analyses results and assumed as the engineering demand parameters (EDPs) of the present study. A total of twenty scalar intensity measures (IMs) were then selected among common parameters that can be extracted directly from ground motion time series (e.g. PGA; PGV, etc.), quantities obtained from response spectra (e.g. spectral acceleration), parameters obtained through integration over a defined period range (e.g. Spectrum intensity) and more advanced scalars which recall relevant aspects, such as spectral shape, that the traditional IMs cannot consider. Each pair (EDP, IM) was investigated in terms of correlation in the transformed space by means of ordinary least squares and robust linear regression analyses, assuming the validity of the power model [10]. The effect of using vector-based IMs (3 in total), obtained coupling traditional intensity measures, for the seismic response prediction was also examined, through multivariate regression. Efficiency, Practicality, Proficiency and Product Moment Correlation Coefficients were the adopted parameter that allowed the identification of the optimal IMs.

3 CONCLUSIONS

The main goal of the presented work was to study and compare the predictive capabilities of traditional and advanced intensity measures (IM) for the response of reinforced concrete bridge populations. To accomplish such objective, a significant number of nonlinear dynamic analyses was carried through continuum interaction between Matlab and OpenSees. Engineering demand parameters were then extracted and their correlation with twenty IMs was scrutinized in terms of efficiency, practicality, proficiency and Pearson Product Moment Correlation Coefficient. The analysed bridges were part of typical Italian bridge portfolio, randomly generated according to Latin Hypercube Sampling method procedure. To assess the performance of the different IMs, thirty different input ground motion records, at seven intensity measure levels, were selected according to the Conditional Spectrum method, compatibly with the seismic hazard of Molise region. The results have demonstrated the existence of a good correlation between a proper IM and EDPs for a population of randomly generated bridges. Fajafar Index (Iv), peak ground velocity (PGV) and root mean square velocity (vRMS) turned out as the optimal intensity measures, when a population of bridges is considered. Commonly employed IMs, such as peak ground acceleration (PGA) and spectral acceleration (Sa), exhibited nevertheless acceptable performance. The final choice of the intensity measure to use will thus rely on personal judgment as well as the balance between enhanced performance and available hazard specific data.

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