SHAKING TABLE TESTING OF NONLINEAR ELASTIC MOMENT RESISTING FRAMES

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ABSTRACT :

A scaled physical model of a nonlinear elastic moment resisting frame is designed and tested both statically and dynamically with the aim to validate a numerical model as well as confirm the global properties of this type of structures. The nonlinear stiffness of this type of structures is verified through a static pushover test. Dynamic properties such as the amplitude dependent of the natural frequency are identified. Equivalent viscous damping properties of the physical model are evaluated for a set of different amplitudes through free vibration tests. The data retrieved during physical experiments is used to update a distinct element numerical model of the physical test frame.
The physical model is tested under a large number of real ground motion time histories to evaluate the robustness of the structural form. Selections of these tests are run through the updated numerical model to estimate how well the numerical model emulates the physical model. An example of a time history analysis is presented, showing a good comparison between the physical model and the best numerical model. The best numerical model manages to estimate quite well the amplitude and time of the peak response of the structure to the ground motion.

KEYWORDS: Damage resistant, Self-centring, Nonlinear elastic, Post-tension, Joint, Distinct element

1. INTRODUCTION

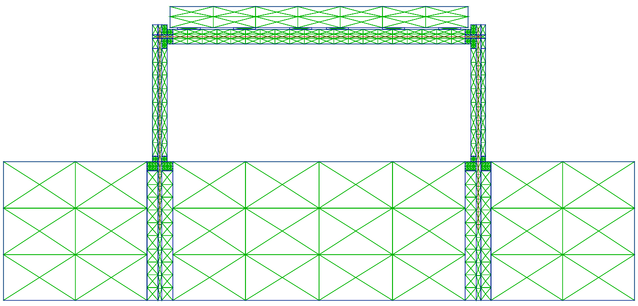
Earthquake resistant design as practiced to day, for example in USA and Japan, has succeeded in reducing the number of casualties during large seismic events however the economical losses are still vast. An example of an recent event is the Northridge earthquake in 1994 with only 57 casualties but around $50 billion in economical losses (Porter, 2006). The financial losses during large seismic events have pushed the engineering practice towards damage resistant structures. Nonlinear elastic moment resisting frames, where deformations are localized in joints between structural elements, remain undamaged despite large deformations.
Although nonlinear elastic moment resisting frames have been researched extensively over the last 15-20 years, there is still uncertainty regarding how these structures will behave under dynamic loading. Large numbers of experiments on confined beam-column assemblies, tested under cyclic loading up to failure, have been carried out. Phase III of the PRESSS project dealt with Pseudo-dynamic testing on a 5 story 2 bay model building at 60% scale. All the experimental results carried out for this class of structures were very promising, only a minimal amount of damage was normally observed up to design level (Priestley, 1999).
To develop a comprehensive understanding of the dynamics of this class of structures a scaled physical model of a frame utilizing this type of connections between structural elements is constructed. The physical model is utilized to determine the characteristics of the structure under static and dynamic loading. The data retrieved during the physical experiment is then used to update a distinct element numerical model of the physical model.
Finally the validity of the numerical model is explored during nonlinear time history analysis where the input motion is a real earthquake time series.

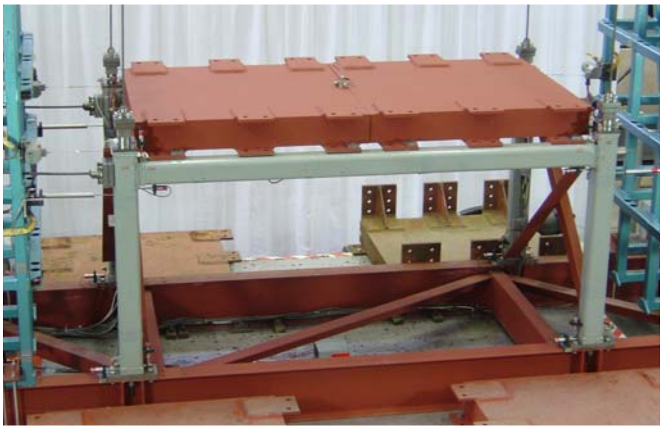
2. EXPERIMENTAL MODELS

Numerically it has been identified that nonlinear elastic frames, having dry joints, where elements are connected solely through post tension, exhibit some typical features of nonlinear dynamic systems (Oddbjornsson, 2007). The numerical modelling was done utilizing UDEC (Itasca, 2004) a commercial distinct element software. Properties such as damping, joint stiffness and friction had to be determined from available literature. From this numerical modelling, properties such as tendon tension force changes, joint shear displacements and joint contact area were identified to increase the understanding of the mechanics of this type of structures. The frame force-deflection curve was identified to assess the global stiffness of this type of frame. A scaled physical model of a nonlinear elastic moment resisting frame is designed and built. The purpose of this was to confirm the existence of various features identified in the numerical simulations. The frame selected for experimental modelling is a single bay single storey portal frame. The numerical simulations suggested an extremely complex system that included both joint opening and sliding as well as variations in tendon loads. This intricate behaviour resulted in features such as nonlinear resonances, quasi-periodicity and perhaps chaos. Thus, it was considered prudent to fully understand a simple physical model before progressing onto more complex structures. The mechanics of this scaled model are explored statically and dynamically through both physical and numerical experiments.

2.1. The Physical Model

The physical model is a quarter scale model based on a typical rather light weight portal frame building prototype; where the scaled model represents a single bay and a story from that building. The prototype to model scaling follows artificial mass simulation modelling rules (Harris, 1999). The model frame bay width, column centre to column centre, is 2100mm. The height from base to centre of the beam is 900mm. The 2t mass on top of the two frames represents the scaled applied load on the beams. The detailed design of the model building was done according to design guidelines presented as a part of the PRESSS research program; this structural form was first proposed by (Stanton, 2002). Following the design guidelines assuming 100mm initial contact height, a tendon tension and cross section of 115kN and 93mm^2 for beam tendons and 64kN and 52mm^2 for column tendons are determined. The elements of the model frame are made from 100x100x10mm steel square hollow section rather than concrete as in the PRESSS building. This material selection does not influence the global behaviour of the system since deformations are restrained to the joints between elements and elements remain elastic through out the design range. The initial contact area at each joint is 100x100mm in size and the contact surface is steel to steel. Before assembly all the contact areas where ground in the same way to generate as uniform contact properties as possible.





(a) (b)

Figure 1 Experimental models, (a) Numerical model, (b) Physical model

2.2. The Numerical Model

A distinct element numerical model of the small scale test frame is generated in UDEC, commercial distinct element code software. The numerical model, consisting of 4 deformable blocks connected together with 3 cable elements, represents one of the two frames of the scaled physical model. The mass on top of the numerical model is set to half the total mass of the physical model and the tendon properties are selected as equal as those of the physical model. The elastic modulus of the beam and the columns of the numerical model is adjusted such that axial and moment stiffness are the same as in the physical model. Unknown parameters of the system such as joint shear and normal stiffness as well as frictional properties and global damping are selected from literature as a first guess and then updated to fit the experimental data.

3. SYSTEM EXPLORATION

A simplified analytical model of this type of structures is essential to explore the global dynamics of complex structures utilizing these joints. A distinct element model although capable of modelling this type of structures is just too slow. The computational analysis (on a 3400Mhz Pentium 4) takes about 2-3days. This is for a time history analysis of just 25s on this single portal frame model. To generate a simplified analytical model of this type of joints and jointed structures, it is essential to explore the system mechanics. A validated distinct element numerical model as well as a physical model are necessary to understanding the mechanics of these joints. In the following two subsections the mechanics of the system are explored both statically and dynamically and unknown parameters of the numerical model adjusted to fit the physical model data.

3.1. Quasi Static

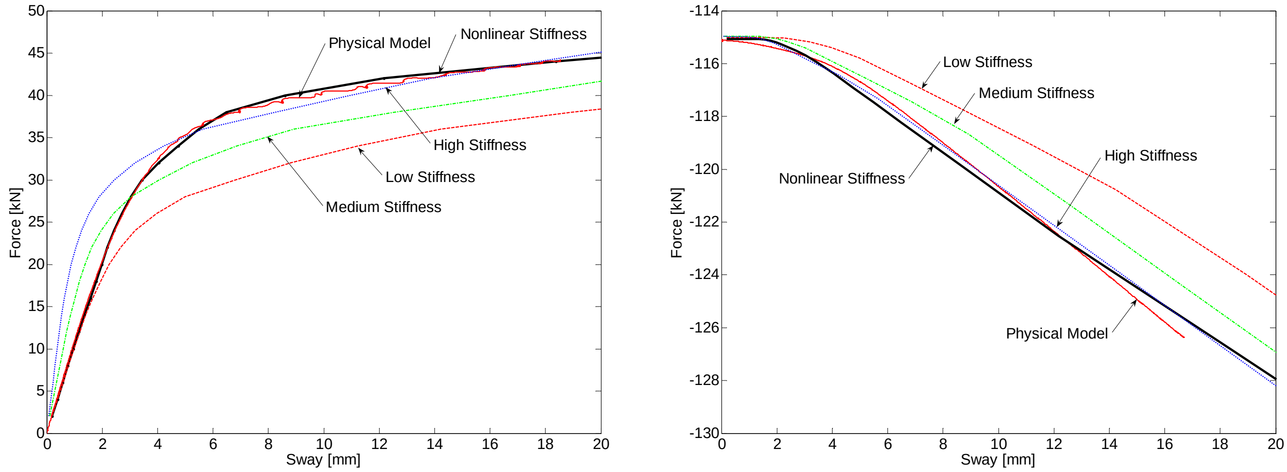
The nonlinear force deflection characteristics of the frame are determined through a quasi static pushover test for both the numerical model and the physical model. The physical model is pulled back by a hydraulic actuator while the force and deflection changes are monitored. The numerical model is pulled back by an incrementally increasing load and the relative displacements recorded. This is done while varying unknown parameters such as joint stiffness and frictional forces. By trial and error it is determined that the key parameters controlling the force deflection properties of the frame are initial tendon tension, tendon stiffness, contact height and joint normal stiffness. Out of those parameters the only unknown is the joint normal stiffness initially selected as 10 times the stiffness of adjoining elements as recommended in the UDEC manual (Itasca, 2004). By adjusting the joint normal stiffness it is possible to match the numerical data to the physical data retrieved during physical pushover tests, the constant joint normal stiffness values explored are listed in table 3.1.

Table 3.1 Joint normal stiffness

Model	Joint Normal Stiffness [GPa/m]
Low Stiffness	14
Medium Stiffness	25
High Stiffness	45

It is obvious from figure 2 (a), where the nonlinear pushover curves for the physical model and the numerical model are displayed, that the numerical model with low joint normal stiffness fits the physical model well at low amplitude but is too soft at high amplitude. The numerical model with the high stiffness fits well at high amplitude but is too stiff at low amplitude, the medium stiffness value generates the best fit with a constant stiffness but it is still not a good fit. A review of papers dealing with contact normal stiffness suggests that contact normal stiffness are nonlinear, softer initially and then harden up (Krolikowski, 1991). By introducing a nonlinear joint normal stiffness that is equal to the low stiffness under low stress and shifts to the high stiffness value under high stress, it is possible to fit the pushover curve from the physical experiment almost perfectly. The beam tendon tension force change is monitored for both numerical and physical models, the tendon force sway relation is displayed in figure

2 (b). The best fit numerical model for the beam tendon tension force is the one with the nonlinear joint normal stiffness, same as for the pushover curve.



(a) (b)

Figure 2 (a) Pushover curve, (b) Beam tendon tension force

3.2. Dynamic

Dynamic free vibration tests are utilized to determine the natural frequency amplitude as well as the damping amplitude properties of the system. A free vibration test is conducted on the physical model by pulling it back by a hydraulic actuator attached by a reduced diameter stud working as a fuse, breaking when the structure reaches a certain displacement, depending on the stud diameter. The natural frequency of each half cycle is then evaluated by determining the time between zero crossings of the displacement signal and assigned to the absolute maximum response of the system during that half cycle. Figure 3 (a) shows the frequency amplitude relationship of the physical model, as expected the frequency is lower at high amplitude i.e. when the system is softer.
The equivalent viscous damping ratio for the system is determined through logarithmic decrements, determined from the change in maximum response of the system during each cycle. To reduce the influence of noise, the average damping ratio over 3 cycles is determined as

$$\zeta_i = \frac{2\pi}{3} \ln\left(\frac{u_i}{u_{i+3}}\right) \tag{3.1}$$

where ζ is the equivalent viscous damping and u is the maximum amplitude. This damping ratio value is assigned to amplitude

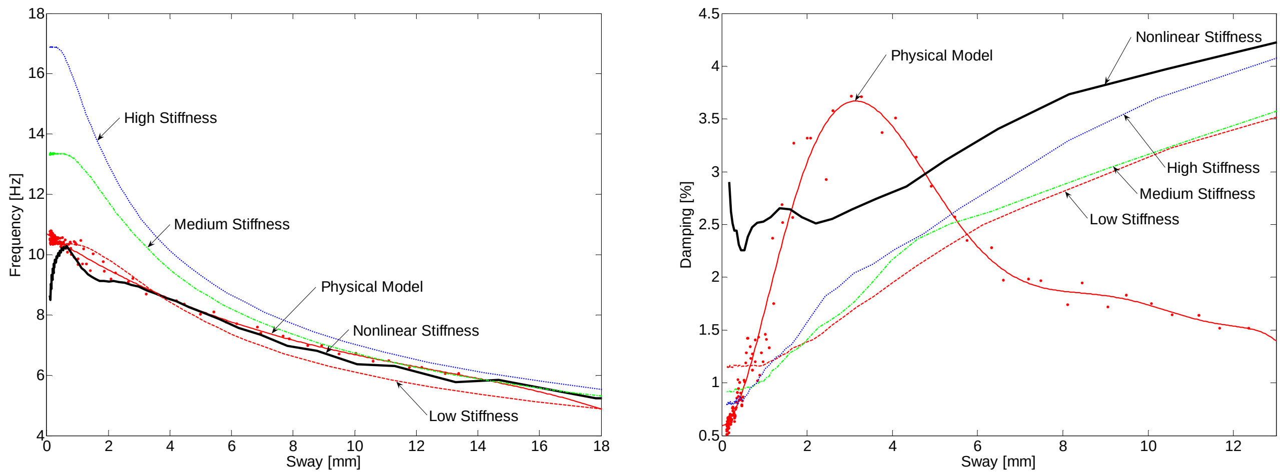
$$U_i = \frac{\sum_{j=0}^{3} u_{i+j}}{4} \tag{3.2}$$

The relationship between equivalent viscous damping ratio and amplitude is displayed in figure 3 (b). The relationship is rather complicated with maximum damping at medium amplitude and lower damping at high and low amplitude.
To get the damping properties of the numerical model to emulate the physical model as well as possible it is necessary to understand the cause of damping in the numerical model. The numerical model dissipates energy through frictional movements of the joints as well as with additional Rayleigh numerical damping. Mass

proportional Rayleigh damping is the only practical options, stiffness proportional is too slow. The problem with the Rayleigh damping is that a damping frequency as well as damping ratio has to be selected. Through trial and error, a mass proportional Rayleigh damping ratio of 2% at 10 Hz is selected. A numerical free vibration test, where the frame is loaded up quasi statically and then put into a dynamic free vibration, is conducted for all the numerical models having different joint normal stiffness. The data from the numerical free vibration tests is processed in the same way as the physical experimental data; the results are displayed in figure 3.

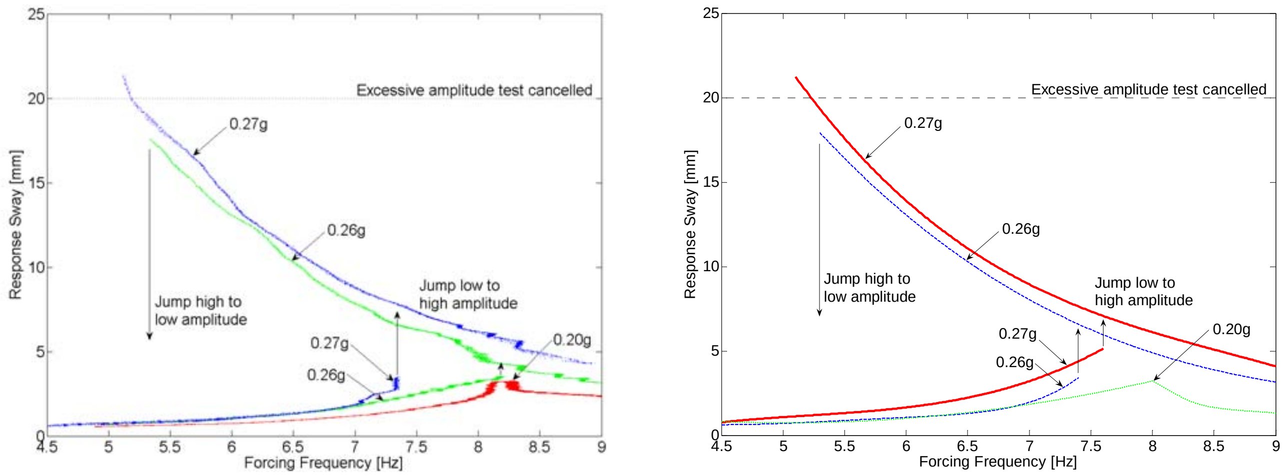
As would be expected the results for the natural frequency amplitude relationship, of the numerical model with the nonlinear contact stiffness, fits the physical model data best. The damping properties of the numerical model are not capable of modelling the complex damping properties of the physical model properly. Complicated damping properties of the physical model, such as friction between tendon and element as well as the friction between wires of multi strands, are not modelled in the numerical model.

The nonlinear resonance response curve of the physical model is determined though a sine frequency sweep. The ground acceleration is kept constant and the frequencies are swept from low to high and vice versa. Figure 4 shows the nonlinear resonance response curve of the physical model for ground acceleration of different amplitudes. The plot shows a jump from upper to lower branch of the resonance response curve and vice versa. This phenomenon is a well known artefact of nonlinear softening systems (Thompson, 2002).



(a) (b)

Figure 3 (a) Frequency amplitude characters, (b) Damping amplitude relationship



(a) (b)

Figure 4 Resonance response curves (a) Raw experimental data, (b) Fitted experimental data

4. GROUND MOTION RESPONSE

The dynamic behaviour of the physical model is explored for a wide range of real ground motion time histories. With the aim of evaluating the performance as well as the robustness of this structural form these ground motions are applied at different amplitudes. An example with a time history of the ground motion from Loma Prieta (1989) earthquake is presented here.

4.1. Physical Experiments

The performance of the physical model to ground motions is monitored during hundreds of tests at different amplitudes. The overall performance of the model is good, with no degradation during tens of hours of testing. A failure of a single wire in a multi strand happened during testing. This did not influence the global dynamics, it was even possible to stop the shaking and replace the multi strand without collapse or permanent damage. Joint sliding effects during a seismic time series have negligible influence on the mechanics of the system.
An example of the response of the physical system to the Loma Prieta ground motion at 105% amplitude, shown in figure 5, is presented. The absolute maximum relative response of the physical model and the time at which the response occurs is presented in table 4.1. The frequency content of the relative response to the earthquake is presented in figures 6 and 7 (a).

Table 4.1 Model response to ground motion

Model	Absolute Maximum Response [mm]	Time [s]
Low Stiffness	26	10.91
Medium Stiffness	22	10.89
High Stiffness	18	10.88
Nonlinear Stiffness	15	10.73
Physical Model	14	10.71

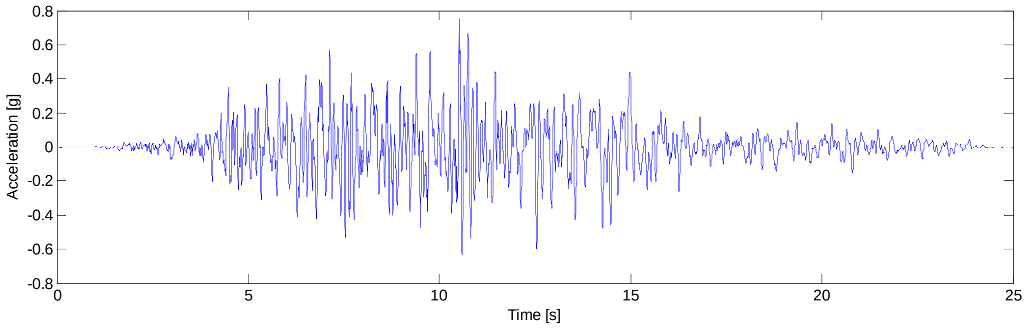


Figure 5 Table ground motion Loma Prieta

4.2. Numerical Experiments

In section 3, the validation of the numerical models has been explored for both quasi static as well as simple free vibration dynamic tests. It has been shown that the numerical model with the nonlinear joint normal stiffness emulates the physical model quite well but not perfectly. To evaluate how much the numerical and physical models differ for stochastic ground motion, numerical time history analyses are conducted.
The Loma Prieta ground motion time series presented in figure 5 is applied to each of the four numerical models and the relative response of the system to the input motion determined. This is a very time consuming process with each time history analysis taking about 3 days. The absolute maximum response and its temporal ordinate is presented in table 4.1. The best numerical model is the one with the nonlinear joint normal stiffness as expected. The frequency content of the relative response to the earthquake is displayed in figures 6 and 7 (a) for each of the

models. As before the model with the nonlinear contact stiffness performs best. To determine how well the response of the numerical model matches with the physical model the magnitude squared coherence between the time series is determined. Figure 7 (b) displays the coherence between the physical and the numerical models. The coherence is highest for the numerical model with the nonlinear normal joint stiffness.

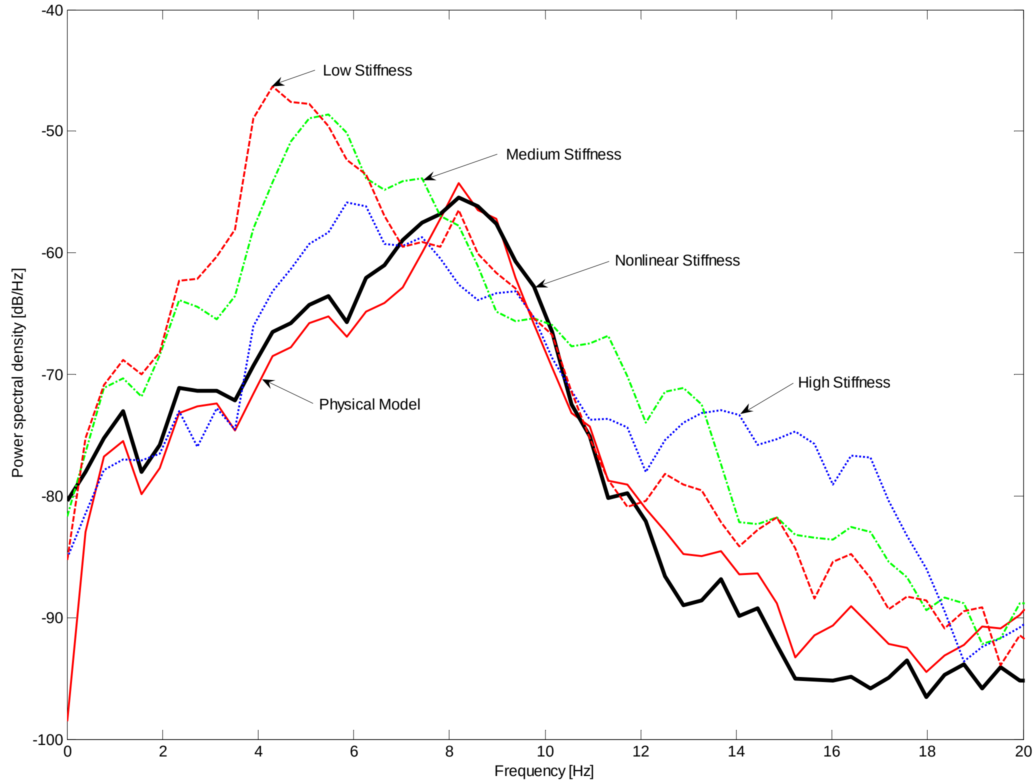


Figure 6 Power spectral density of each model

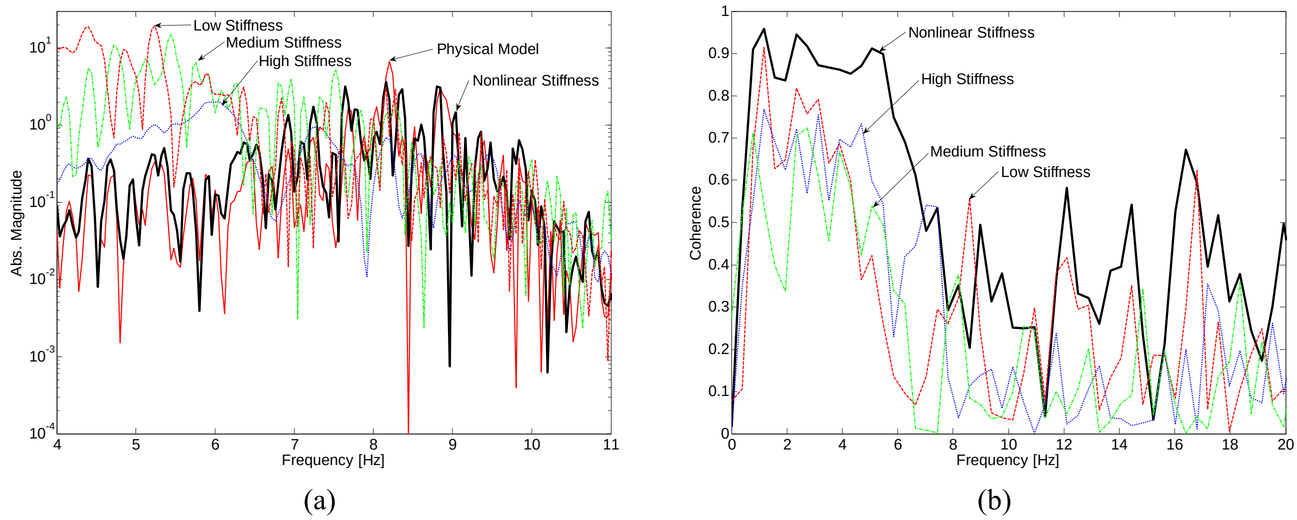


Figure 7 (a) Frequency response to ground motion, (b) Coherence between physical and numerical models

5. CONCLUSIONS

A scaled physical model, of a nonlinear elastic frame assembly, is designed and tested both statically and dynamically. Thus, the underlying mechanics observed numerically have been qualitatively verified. The test

results from the physical model are then used to update and evaluate a distinct element numerical model of the physical model. First the numerical model is updated by comparing response under a quasi static pushover test and a free vibration test. When the numerical system parameters have been adjusted to represent the physical model as well as possible, the system is evaluated under stochastic ground motion.
The quasi static pushover test of the physical model confirms the nonlinear softening stiffness characteristics of the system. The nonlinear elastic softening system exhibits two simultaneous coexisting solutions over a wide range of frequencies. The system can jump between the high amplitude and the low amplitude solutions. This behaviour is a well-known feature of a softening system, such as the Duffings oscillator (Thompson, 2002).
Through a free vibration test of the physical model the change in natural frequency with response amplitude is verified. From the free vibration data the equivalent viscous damping of the system is also extract and the damping relationship with amplitude determined.
The quasi static mechanics of the numerical model with the nonlinear normal joint stiffness fit the physical data very well, suggesting that the joint normal contact stiffness of the physical model is definitely nonlinear. Further physical tests are required to fully explore this nonlinear joint contact stiffness. Since the frequency amplitude relationship is primarily controlled by the stiffness of the system it is no surprise that the numerical and physical data fit well. The nonlinear contact function used in UDEC was bi-linear. It is suggested that an even better fit might be achieved by introducing a smoother nonlinear contact stiffness function.
The numerical and physical models differ most in damping representations. Since we are bound by the damping model available in the distinct element software a match is unlikely to be achievable. Even so, the results presented, in figures 6 and 7 show qualitatively similar responses. The peak response measures in table 4.1 also show a good match.
Future publications introducing a simplified analytical model of these types of structures shall be presented; these models shall be based on nonlinear system identification of experimental data.

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