Time History Analysis is easy, and you should do it: Here's how..

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ABSTRACT

Time history analysis is generally regarded as a highly specialised and deeply technical process. There is also a growing divide between different schools of thought around how analysis should be done. On the one hand, older- perhaps wiser- engineers believe all the good work is done by the human brain with simple pencil and paper calculations. In the other camp, typically younger, digitally native engineers believe (often without consciously knowing it's a belief) that computers can and should do the vast majority of the calculation work to solve complex problems. The author believes that the self-fulfilling prophecy that Nonlinear Time History Analysis (NLTHA) is too hard to learn does not have to come true. And that one does not have to choose between the simple and the complex, but rather a good process does not rely on any one method - mistakes can be made anywhere- but that a systematic process with checks and balances is best. While this approach could be used by anyone, it is primarily for use in design offices. The fundamental goal of this paper is to stress that the goal is good analysis to enable good design, and offers a pathway to achieve this.

1   INTRODUCTION

An often overlooked fundamental about seismic design is that it's *all* time history analysis. From the original 0.1g design coefficient derived from recordings of PGA, to Housner's torsional spring derived spectra (Refer Figure 1, Housner, 1997), which rather neatly explained why medium height buildings performed worst in the seminal earthquakes of the early 20th century, to the k_μ force reduction factor, based on a casual observation of Newmark's, actual observations of dynamic behaviour underpin almost all of what we do.
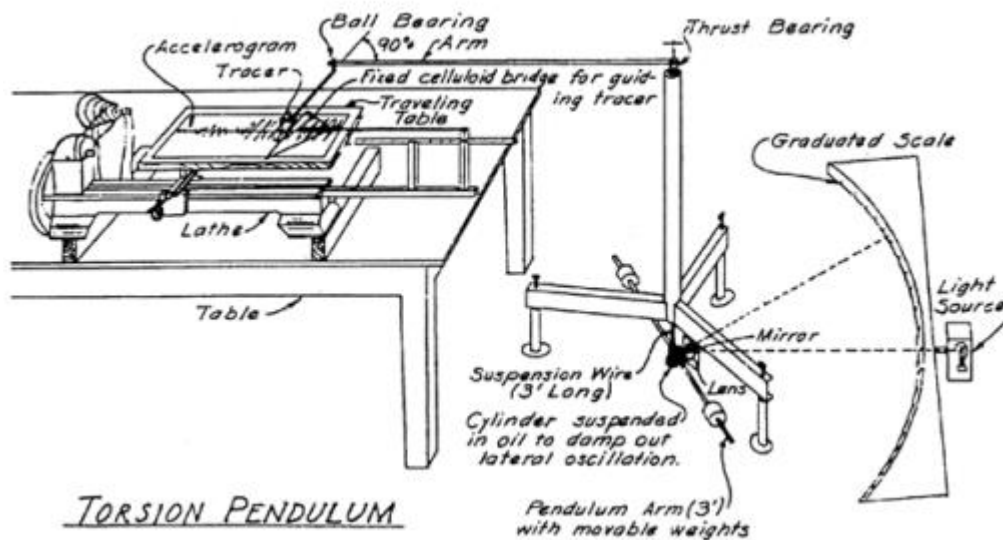
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Figure 1: Housner's Torsion Spring Spectra Generating Apparatus (Housner, 1997)

On the other hand, the great professor of structural engineering dynamics Anil Chopra published a paper following the Northridge Earthquake that declared that buildings are somewhere between 3 and 12 times as strong as what we design them for (Juan Carlos de la Llera, 1998). The comparison with lowest allowable design strength in that paper may be contentious, but this paper's author's own forensic work of buildings following some of New Zealand's recent earthquakes indicates a value in the 2 to 4 range is credible, and only about half of the difference can be readily explained. We are faced with the fact that maybe we do not fully know what we are doing.

We are also faced with increasing demands on the limited resource of engineering time. The sheer amount of documentation, particularly calculations, expected by Peer Reviewers and Territorial Authorities has increased significantly over the past 10 years. We need to think more carefully about what the right places are to spend this very valuable resource. We also need to think very carefully about what a design office can reasonably be expected to produce.

Critical to this is to keep analysis simple and design-focused, and also to demystify complex analysis and make it more readily accessible – a goal is to supplant modal response spectrum analysis, which one of its pioneers is skeptical of (Wilson, 2015), with simple time history analysis.

2   THE SUGGESTED PROCESS

2.1 Barriers to entry

There are many barriers to entry for NLTHA, so lets just begin. Record selection is intimidating, and a lot of people do not even know where to start. For most sites, a significant portion of hazard is due to crustal earthquakes. The University of California has compiled a large database of such records which is easily searchable: https://ngawest2.berkeley.edu/. For Wellington, a common Vs30 is about 250m/s https://quakecoresoft.canterbury.ac.nz/vs30/. Combining these give a collection of records like the below:

Table 1: Sample Crustal Earthquake Records for Wellington Vs30 = 250 m/s

RSN	Earthquake Name	Year	Magnitude	Mechanism	R_{rup} (km)	V_{s30} (m/s)
6	Imperial Valley-02	1940	6.95	strike slip	6.09	213.44
179	Imperial Valley-06	1979	6.53	strike slip	7.05	208.91
723	Superstition Hills-02	1987	6.54	strike slip	0.95	348.69
776	Loma Prieta	1989	6.93	Reverse Oblique	27.93	282.14
803	Loma Prieta	1989	6.93	Reverse Oblique	9.31	347.9
1045	Northridge	1994	6.69	Reverse	5.48	285.93
1085	Northridge-01	1994	6.69	Reverse	5.19	370.52
1101	Kobe_ Japan	1995	6.9	strike slip	11.34	256
1503	Chi-Chi_ Taiwan	1999	7.62	Reverse Oblique	0.57	305.85
4847	Chuetsu-oki_ Japan	2007	6.8	Reverse	11.94	383.43
5823	El Mayor_ Mexico	2010	7.2	strike slip	19.47	242.05
6897	Darfield_ New Zealand	2010	7	strike slip	8.46	295.74

In the lower north island, the Hikurangi subduction zone also produces significant portions of the hazard (https://nshm.gns.cri.nz/Disaggs). Again UCLA has compiled a great searchable resource: https://www.risksciences.ucla.edu/nhr3/nga-subduction/gmportal. Again, a collection is below:

Table 2: Sample Subduction Earthquake Records for Wellington Vs30 = 250 m/s

Record Sequence Number	Earthquake Name	Year	Magnitude	Rjb (km)	Vs30 (m/sec)
NGAsubRSN4000092	Tohoku	2011	9.12	55	289
NGAsubRSN4028572	Tokachi-oki	2003	8.29	42	255
NGAsubRSN4032460	Kushiro-oki	1993	7.59	-	279
NGAsubRSN4040371	Tohoku	2011	9.12	23	365
NGAsubRSN6001799	Le Maule	2010	8.81	-	303
NGAsubRSN6001813	Le Maule	2010	8.81		278
NGAsubRSN6002200	Iquique	2014	8.15	28	326
NGAsubRSN6002203	Iquique	2014	8.15	32	336
IBR 018 Tohoku	Tohoku	2011	9.12	70	285

Look at 'disaggregations' (the technical term for what type of earthquakes make up your hazard) using the period of your structure to check the percentages of hazard (Wellington at 1.5s is about 50/50 for subduction/crustal), pick some records and lets go. 11 records is the current standard in the US.

We can now make a model. The simplest model possible is a mass on a stick (also known as a single degree of freedom, or more colloquially as a lollipop). In ETABs, this would be a single point mass, connected to a rigid base support by a single 'link', the 'wen plastic' property being one of the most versatile. You'll end up with something which looks like this:

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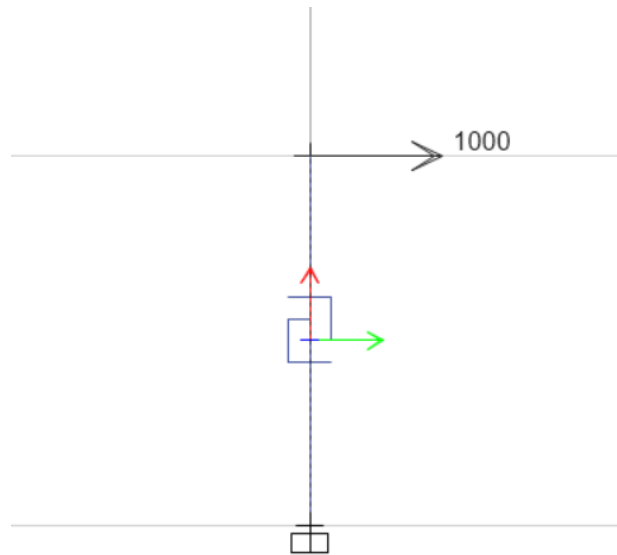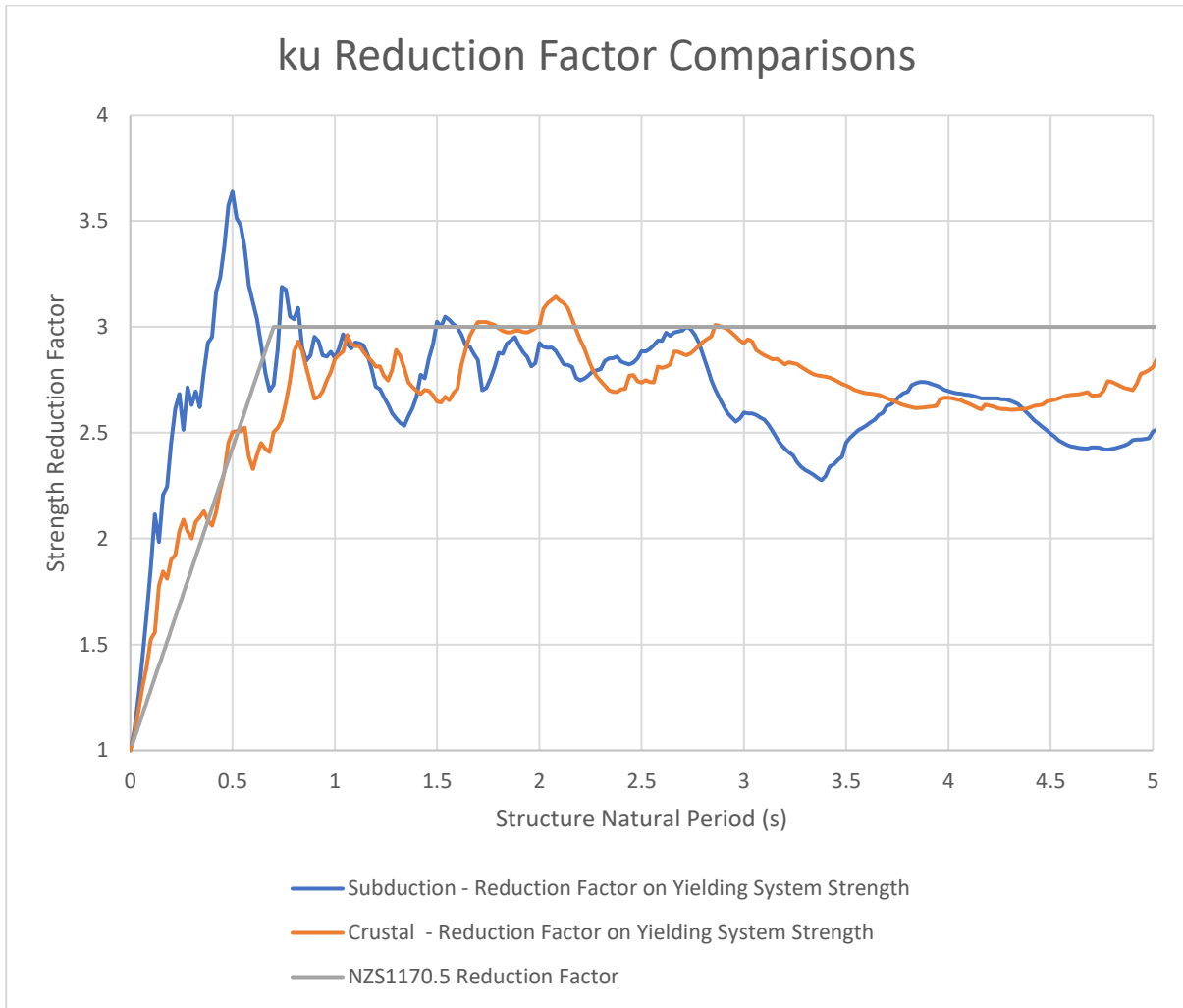
Figure 2: Simple SDOF Stick Model

Don't be put off but the simplicity of this, and don't be put off by the fact that it doesn't 'look' like your building- this is the job of architectural renders. The author has in many cases got much more *accurate* (if less precise) results from models like this than from certain models where each beam and column is specifically 'drawn'. For example, the basic dynamics of a regular, well distributed base isolation system (where the centre of mass and strength are aligned) with a stiff superstructure can be well captured by representing it this way (though you may need 3D models to capture accidental torsion effects)

Once we have a simple model and some records, we can look at some simple phenomena. An interesting one to look at is the 'equal displacement theorem', the basis for the fact that $k_\mu = \mu$. This 'theorem' is controversial, though there is some support for it in recent literature within some bounds (Bozorgnia, 2010). As a rule of thumb, it is useful. As in the paper mentioned, at large earthquake magnitudes and high ductilities, elastic, perfectly plastic systems stop 'obeying' the 'rule'. That paper doesn't look at why this happens, but if you look at what is actually going on physically, you get some clues. At large magnitudes, the ground displacements are large, and a structure will tend to wander off its baseline, so the maximum displacement actually builds up over multiple cycles. With a small restoring force (2.5% of the elastic stiffness, which is common for steel), the structure will follow the ground more closely, and the rule is a decent match. These graphs were produced using a program called Seismosignal, which is well set up for processing lots of such calculations, but you can check individual period/ strength combinations and get the same results.

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ku Reduction Factor Comparisons

Legend:
- Subduction - Reduction Factor on Yielding System Strength
- Crustal - Reduction Factor on Yielding System Strength
- NZS1170.5 Reduction Factor

2.2 Dynamics of Multiple Degree of Freedom systems, a glance

If we add a second mass on top of the first, we have a 'multiple degree of freedom' system, and the dynamic complexity starts to ramp up. There's obviously not time in this paper to look at all the complex issues around torsion, vertical irregularity, dynamic magnification and so on, but a useful guiding principle is to understand what basic type of multi degree of freedom system you have- shear or flexural. This terminology refers to how a building deforms elastically- through pure shear translation, or by bending (like a pure cantilever). The dynamic properties are different, as below. Confusingly, a moment resisting frame is dominated by shear translation, as the beams are relatively flexible, and a shear yielding eccentrically braced frame is more 'flexural', as the braces make it stiff in shear and it deforms largely by column lengthening/ shortening. Either way, it is useful to make a model, then look at ratios between modes and between modal participations, as this will play significantly into dynamic behaviour.
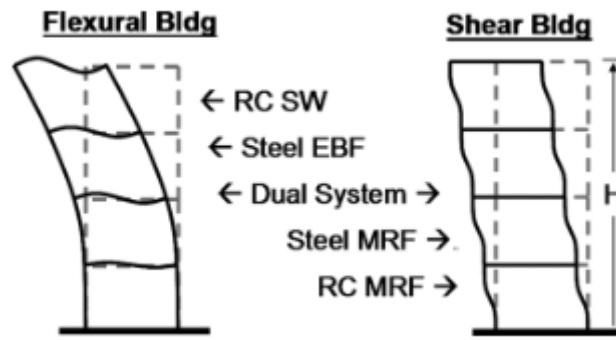
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Figure 3: Graphical Representation of Shear and Flexural Building Deformation from (Derek Skolnik, 2011)

Table III. Frequency ratios

Ideal building	Number of storeys	ω_2/ω_1	ω_3/ω_1	ω_4/ω_1	ω_5/ω_1
Shear building SB-CCS	3	2·80	4·05	—	—
	8	2·97	4·84	6·54	8·03
	14	2·99	4·94	6·84	8·65
	20	2·99	4·97	6·92	8·83
	30	3·00	4·99	6·97	8·93
	50	3·00	5·00	7·00	8·99
Shear building SB-VCS	3	2·45	3·87	—	—
	8	2·45	3·87	5·29	6·72
	14	2·45	3·87	5·29	6·72
	20	2·45	3·87	5·29	6·72
	30	2·45	3·87	5·29	6·72
	50	2·45	3·87	5·29	6·72
Flexural building FB	3	5·28	10·9	—	—
	8	6·08	16·1	29·1	43·4
	14	6·20	17·0	32·5	51·7
	20	6·24	17·3	33·4	54·3
	30	6·27	17·5	34·4	56·9
	50	6·27	17·5	34·4	56·9

Table I. Modal participation factors for base shear

Ideal building	Number of storeys	β_1	β_2/β_1	β_3/β_1	β_4/β_1	β_5/β_1
Shear buildings SB-CCS	3	0·914	0·082	0·012	—	—
	8	0·856	0·107	0·035	0·015	0·007
	14	0·838	0·109	0·038	0·018	0·010
	20	0·830	0·110	0·039	0·019	0·011
	30	0·823	0·111	0·040	0·020	0·012
	50	0·818	0·112	0·040	0·020	0·012
Shear building SB-VCS	3	0·857	0·130	0·037	—	—
	8	0·794	0·143	0·054	0·028	0·016
	14	0·775	0·146	0·056	0·030	0·018
	20	0·767	0·146	0·057	0·030	0·018
	30	0·762	0·146	0·057	0·030	0·019
	50	0·757	0·146	0·057	0·030	0·019
Flexural building FB	3	0·742	0·296	0·051	—	—
	8	0·656	0·313	0·109	0·053	0·028
	14	0·636	0·310	0·108	0·055	0·022
	20	0·629	0·308	0·107	0·055	0·033
	30	0·623	0·307	0·106	0·054	0·033
	50	0·619	0·307	0·106	0·054	0·033

Figure 4: Dynamic Properties of Shear and Flexural Buildings (O. A. LÓPEZ, 1996)

2.3 Common Pitfalls

The great Danish physicist Niels Bohr once said that 'An expert is a person who has made all the mistakes that can be made in a very narrow field.' In the author's (structural engineering centred) experience, nowhere is this truer than NLTHA. This paper is intended as a light-hearted exploration of some of the types of mistakes that can commonly be made, how to avoid them, and how to build your knowledge so you can do better designs. We will begin with some principles, then show how they apply with simple real world examples.

The most important step of all is problem definition. With any modelling process within structural engineering, it can be very seductive to get straight into attempting to mathematically modelling the building as quickly as possible. This is cognitively seductive. It feels like if we model as much as possible, we will be as accurate as possible, and jumping right in feels like instant progress. Perhaps ironically, for simple problems this works just fine. For more complex problems, the numbers of connections between elements, the number of boxes to tick and untick, the system behaviour, and the output will get unwieldly. We will lose sight of the reasons for modelling. For NLTHA, fundamentally, the reason to do it is *to model behaviour which isn't well represented in the code*. Viscous damping, structures with restoring force, structures with irregularities, structures where the code has not caught up. We can isolate the particular phenomenon and study it, and still use the code where it is a sufficient predictor.

If the problem needs a complex model, the procedure is: crawl, walk, run. We develop our model in steps, and at each step, the results should be the same, or we know why they have changed. Crawl is usually best as hand checks. For new problems, this can be fun – 'the intoxicating delight of sudden understanding that

mathematics gives, from time to time, to those who love it' (Russell, 1946). In many instances, this is just familiar static checks such as displacement-based design. 'Walk' might talk the form of a vastly simplified model. Run is modelling the seismic system in full.

Even when running, don't forget the many virtues of simple analysis. Keeping it simple allows you to derive principles from your analysis, rather than just compare demand with resistance. It is also much easier to debug something which is simple. If you are having trouble with a model, go simpler.

The process of stepping through analysis carefully also allows a designer to develop an intuitive understanding of how a building works. The human brain has a remarkable ability to understand plasticity, in a way that can be very difficult to tell a computer how to do- Don't expect the computer to do your thinking for you (though it requires discipline not to). Part of this is to NOT expect the unexpected, and not to tell 'just so' stories. If the computer produces a result you do not expect, you should seek to understand why, and test enough hypotheses to have good grounds for believing an updated theory is correct.

Finally, sensitivity analysis is very important. If we are to design well for uncertainty (as we might not know particularly accurately our building's strength and stiffness), we need to vary our assumptions to check that the building is going to behave well under a variety of conditions. This is a tricky step, as the number of elements which can be varied can be vast (soil springs, stiffness of concrete fill to the steel tubes, ground beam cracked stiffness, yielding element strength, and so on). This is where simple models which run quickly are helpful, and an intuitive understanding can allow you to pick the worst-case combinations more quickly.

3 GOLDEN RULES: EXAMPLES

3.1 The most important step is problem definition

Below is a model of a building currently being built. Obviously, the real thing doesn't look a bit like the model, but that is because we only modelled the behaviour in question- how the building performs in torsion. The stiffness of individual frames, higher vertical mode effects, p delta effects, and so on can all be

calculated separately. Indeed, the frame stiffness calculation is difficult, and highly variable based on some basic assumptions, so better off separated.
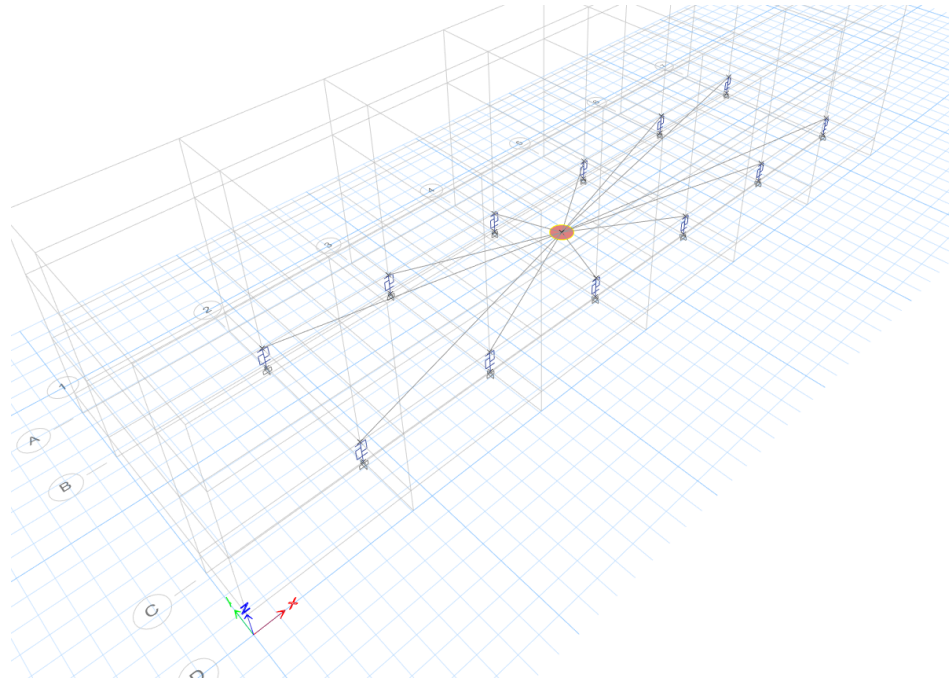

Figure 5: Simple Planar Model for considering dynamic torsion

The building is fairly regular, with a moderate offset between the centre of mass and centre of stiffness, and moderate ductility of 2-3. Due to the aspect ratio, the perpendicular frames have the strength to resist the torsional moment (due to the offset and accidental eccentricity) statically, but when one considers the rotation and due to the aspect ratio, only when the perpendicular structure has reached about 4% drift. If one does a pushover, the structure just pivots about the edge that doesn't yield. Dynamically, when torsionally unrestrained buildings yield at one end, this does not happen, but the rotational inertia holds them in place (M. J. N. Priestley, 2007). This is a purely dynamic problem, so was analysed dynamically to check the effects with multiple assumptions. Remaining items were separated for the purposes of peer review, and resolved outside this process. Effects such as drift modification due to higher modes (k_{dm}) were taken as standard code values. Done simply, this could be done very quickly and the effects could be verified using formulas in the above reference.

3.2 There's always a handcheck: Base Isolator Moat Wall Collision

Pounding is a notoriously difficult problem within structural dynamics. It is a highly dynamic problem, which is very sensitive to assumptions about demand, strength, and stiffness of elements involved.

Dunning Thornthon Consultants worked on an IL4 base isolated project, with the isolators designed to seismic hazard with a 1% probability of exceedance in 50 years (1/5000 return period in historic/ deterministic terminology). To help limit the movement without increasing the building strength – which compromises the ability of the bearings to isolate the building in likely earthquakes- the effect of the moat wall as a last- ditch restraint was examined.

We built as simple a model as possible, with the isolation plane and superstructure represented by single masses, and the collision represented by a gap element and yielding spring. Even this simple model took time. To verify it, a simple hand check was developed based on energy balance of a single half cycle, which turned out to be a quadratic equation. It could predict the NLTHA to within fractions of a millimetre, and allowed us to understand what the controlling terms are, as well as iterate efficiently to come up with the

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most appropriate parameters. In this instance, we found a cantilevered concrete moat wall designed to hold up a mere 1-2m of soil had a limited ability to hold up 4000t of steel and concrete hurtling towards it, but how far through the cycle it hits (the kinetic energy) is important, as is appropriate detailing to ensure superstructure forces are controlled.

The simple analysis drove the design process, and a collision link was added to the full building model to make sure nothing was missed from the simplified analysis.

The result for the building was that we could slow down the building just enough to keep the isolator buildable, while not adding strength which would compromise the performance.
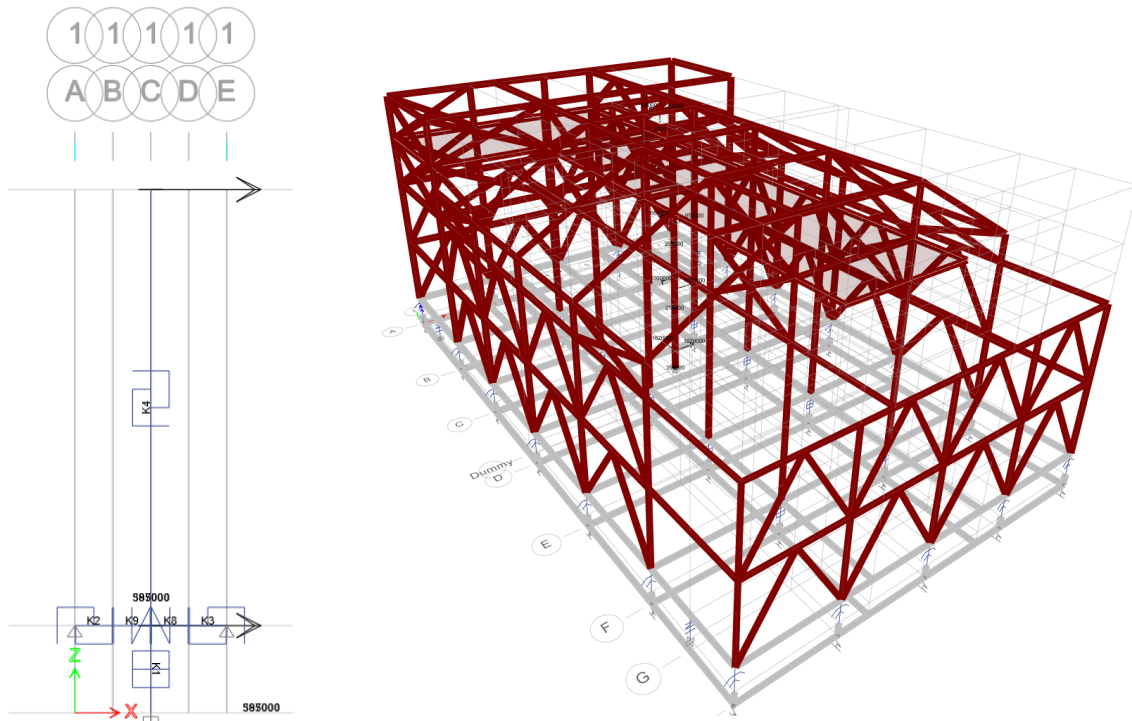
Figure 6: Simple (Left) and More complex (right) models for considering corner pounding

3.3 If you make it simple, you can make rules: Base isolator superstructure ductility

In earlier days of base isolation, it was a common train of thought to design an isolation system, then take the acceleration out of the analysis, divide it by a ductility and design the superstructure with this.

It is important to note that the equal displacement theory is an approximation developed in the 1960s when access to significant computing power was limited, in order to do linear static approximations of inelastic behaviour. It is not an immutable law of nature and casual use of it to reduce the design forces of the super structure in a base isolated building should be treated with deep scepticism.

The period shift implied in the equal displacement theory will not be desirable as it will shift the behaviour into resonance with isolation system. And the inelastic damping will shift from being in the isolators- which have very large displacement capacity and reliable energy dissipation- to the superstructure, which often does not.

A human being isn't born with this knowledge. We had to figure it out. Again, we had an existing model of the whole building. We could have modelled all the ductile elements, thrown some big earthquakes through

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it, and checked whether the hinges lit up green, orange, or red. This would have given us a compliant building, but not necessarily a good one, and we would not have left with much knowledge.

So we did lots of reading, sat with pencil and paper, and eventually when ready ran a very simple model with 2 masses, 2 links, and very clear outputs. The design result is that we adjusted the isolator properties to give them better performance, removed bespoke yielding elements, added some nominal ductility, and saved the client money.

3.4 If there's problems, it's always damping: Base Isolators with cantilevered columns

It this instance, we were designing a building which is base isolated above the ground floor. This was largely a cost driven decision:excavation and tricky foundation construction near the waterfront and a basement below tide level being complex and expensive. This provides a tricky structural problem, as columns have to cantilever up a story and take the large isolator overturning forces, while remaining a sensible size.

We had done simple static checks of the columns to check the drifts were low, but wanted to investigate the dynamic behaviour of the system, particularly if the column was near yield at the base.

It was rather an unpleasant shock, then, that we ran this analysis, and the column displacements were very large – a exemplary instant in time is below:
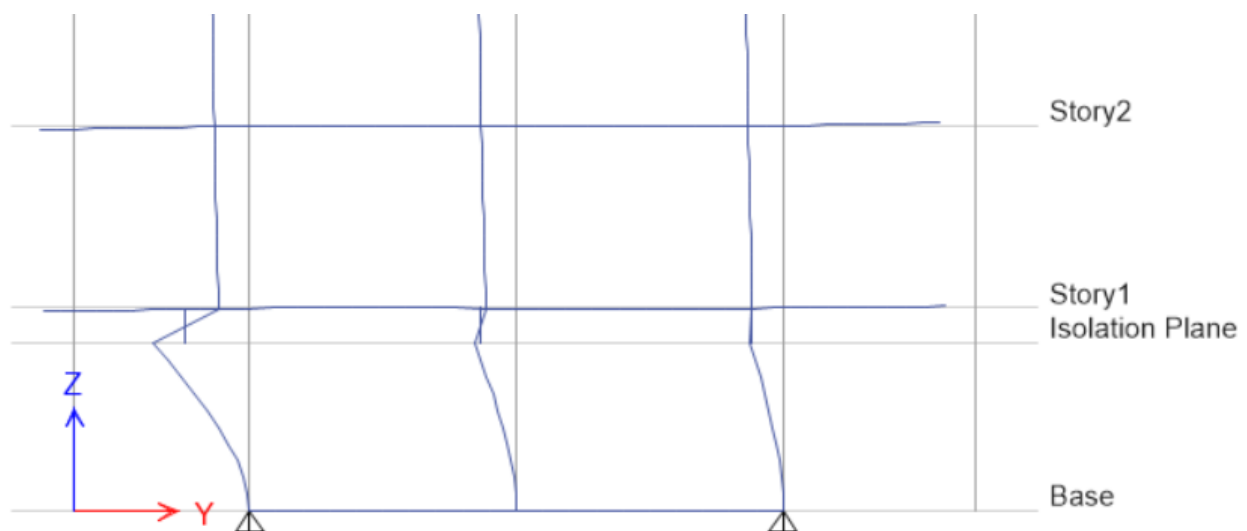

Figure 7: Hidden Damping causing undesirable behaviour

Fortunately, the shock was not particularly long lived- the example above is clearly not in equilibrium. This is not *impossible* – p delta forces could push the column into buckling, but when stepping through the record in time, this is not what happened, but the shape spontaneously happened and force balance with the output forces in ETABS could never prove equilibrium. And when the Rayleigh damping parameters were adjusted, the phenomenon changed substantially. It turns out the very short period of the columns generated a very large (artificial) Rayleigh damping – a few hand calculations could show the approximate value of the force, which matched the missing equilibrium force. The damping was lowered, which made the columns movements much lower, and the actual real behaviour was satisfactory.

3.5 If It's not damping, it's shells

Within seismic engineering, shells are frequently used to simulate the behaviour of reinforced concrete elements in a building – particularly walls. An immediate cautionary statement is that simple shells are elastic elements, with the neutral axis centred in a rectangular section. This is obviously not how reinforced

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concrete works in the ultimate limit state, as the section cracks and the reinforcing yields. A good procedure is to assign piers and spandrels carefully and extract element shears, rather than try to design for a stress. This should always be able to be understood with simple force balance.

Meshing can create problems too. Below is an example where this happened, along with examples of a couple of other rules too. The walls are nicely meshed- to a level where there is sufficient meshing where the frame elements enter the shells to show appropriately smooth contours of force distribution. Unfortunately, when the model was run, Fast Nonlinear Analysis (FNA) could not run at all. Direct integration (DI) ran, but was painfully slow. Turns out the shells were assigned to be membrane elements to keep the analysis simple. When they were meshed, they no longer could support themselves and became a mechanism.
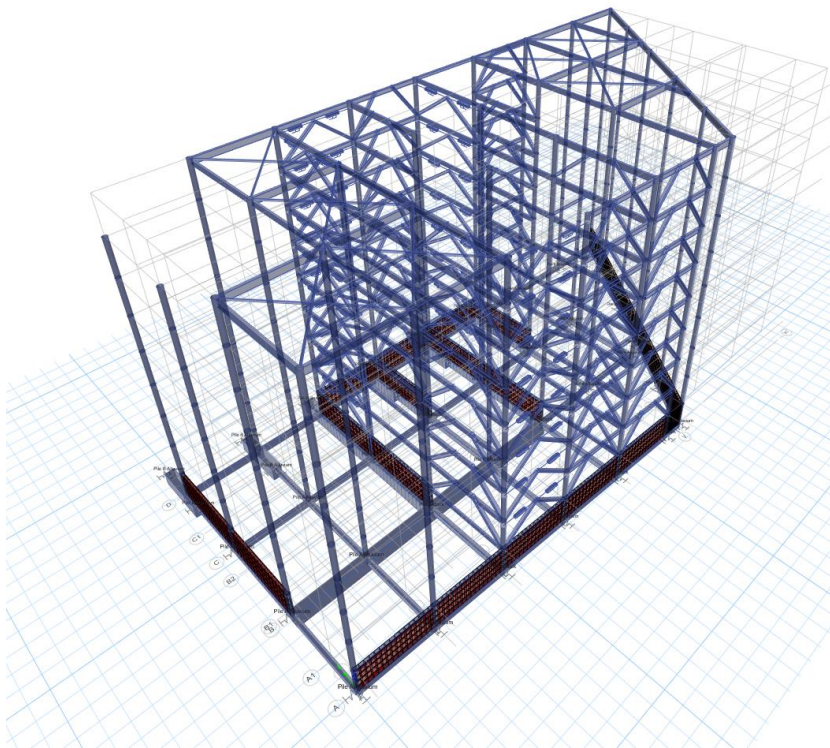

Figure 8: Full Building Model incorporating shell elements

Rules abound in this instance, but a few of them are: Do not tolerate slow models, be careful to revisit your assumptions if your modelling strategy changes, be careful meshing shells. This is also an example of 'don't tell just so stories'. One can invent all kinds of stories about why direct integration works in this instance, and not fast nonlinear analysis. But what's important is to test whether these assumptions are true, look at the analysis run log, and don't just change your opinions to suit what has happened. It is also important that the differences between DI and FNA are in how geometrically nonlinearity works, and how a damping matrix is populated. If you need to go from one to another, you should always arrange a step between where you are careful matching assumptions, and ensuring the results are almost exactly the same.

3.6 If you're having trouble understanding something, go simpler

The building in the figure below is one that actually exists, and it looks pretty similar to the picture too. The model is something of a work of art, built meticulously based on the existing documentation, and surveys of the building. Unfortunately, the structural arrangement is very tricky, with walls at all manner of angles to each other, several different bracing lines of various strengths and stiffnesses, eccentricities, and highly coupled modes. There is also some uncertainties in the founding, which can make a significant difference to

the performance of concrete walls. In trying to work out the right location and amount of strengthening, and to satisfy code requirements that sufficient directions were considered, interrogation of the model became very difficult.
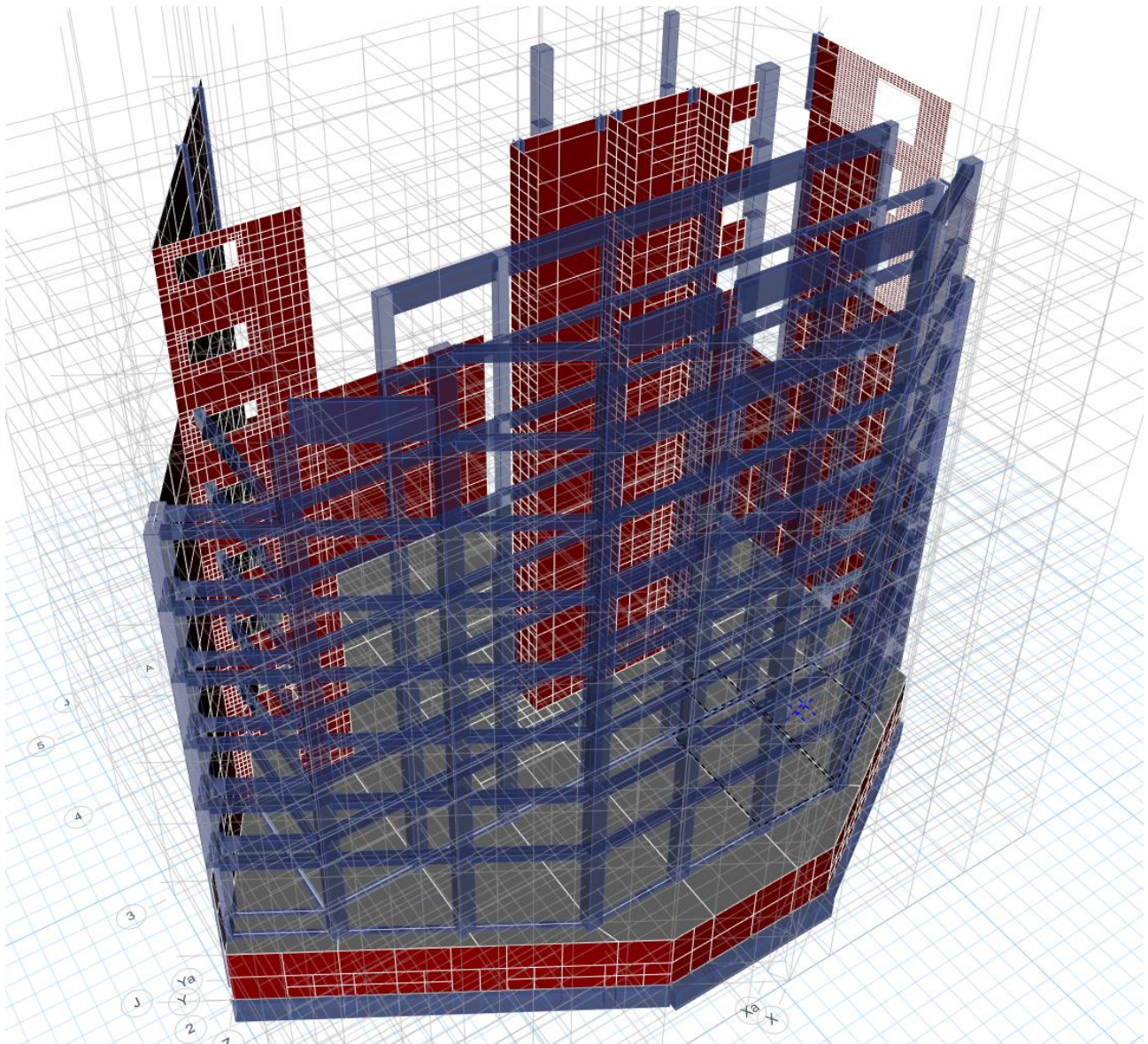

Figure 9: Full Building Model

The solution to this was to go simpler. Much much simpler. We worked out with pencil and paper where the main lines of resistance are, how stiff and strong they are, where the centre of mass is, and where the centres of stiffness and strength are. This allowed us to add strength to the existing structure where an appropriate hierarchy can be easily ensured, as well as ensure that torsional response is as well controlled as possible. We could then roughly predict foundation demands, which were then verified using more complex analysis which included running demands at multiple rotations to satisfy code requirements for buildings without clear orthogonal lateral elements.
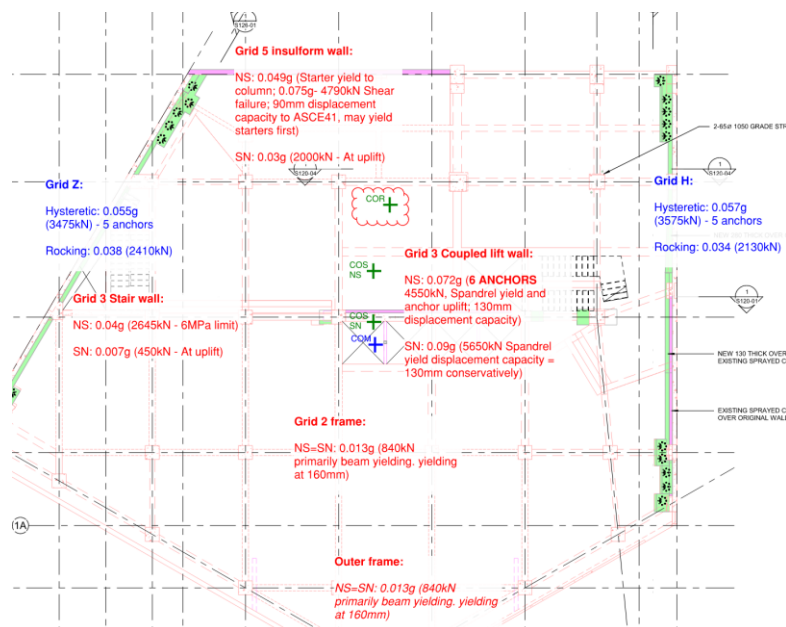
Figure 10: A simpler "model" of the building which is an intermediate step to aid understanding

4 CONCLUSIONS

This paper has stepped through a process to perform simple NLTHA. It is hoped that it has gone some way to demystifying the process for people that have not done it often. If you can start by building simple models, and proving concepts you already know, you can build confidence and complexity on top of this. It has emphasized that simplicity is still very desirable, and that the human designer should always be the one in charge. We have also gone through some common pitfalls in analysis- to give a running start to people new to the process, but to also stress that methodical processes which are understood at each step are best. With practice, this process can be done well, and fast. You should try it.

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