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NONLINEAR DYNAMIC ANALYSIS – THE ONLY OPTION FOR IRREGULAR STRUCTURES

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SUMMARY

The response of buildings to earthquakes is a complex, three dimensional, nonlinear, dynamic problem. Limitations in technology and the depth of our understanding of this problem have lead to the profession developing a number of simplified methods for representing it, most of which disregard one or more of its fundamental aspects: the Linear Dynamic Procedure (LDP) ignores nonlinearity; the Nonlinear Static Procedure (NSP) ignores dynamic effects; the Linear Static Procedure (LSP) ignores both.

In contrast, the Nonlinear Dynamic Procedure (NDP) attempts to fully represent the seismic response of buildings without any of these major simplifying assumptions.

This paper discusses some of the obstacles preventing the widespread adoption of NDP, then presents a number of examples where its uncompromised representation is crucial for satisfactorily predicting the seismic response of structures. Specific case studies include rocking systems, structures with significant stiffness irregularities and existing structures with inadequate seismic resistance.

It is concluded that the NDP is the only universally appropriate method for verifying the performance of structures, and that the traditional reasons preventing its widespread adoption are all but invalid. The profession needs to change their collective mind set, stop resisting and start embracing what technology now enables us to do.

INTRODUCTION

The response of buildings to earthquakes is a complex, three dimensional, nonlinear, dynamic problem. Limitations in technology and the depth of our understanding of this problem have lead to the profession developing a number of simplified methods for representing it, most of which disregard one or more of its fundamental aspects: the LDP, or Response Spectrum Analysis, ignores nonlinearity; the NSP, or Pushover Analysis, ignores dynamic effects; the LSP, or Equivalent Static Analysis, ignores both.

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In contrast, the Nonlinear Dynamic Procedure (NDP), or Time History Analysis, attempts to fully represent the seismic response of buildings without any of these major simplifying assumptions.

Despite the compromises inherent in the simplified analysis techniques, the NDP is used infrequently in the profession. The LDP is perhaps the most common analysis procedure used in the design office for multistory building design. Although not necessarily appropriate, design codes such as the New Zealand Loadings Code, NZS4203:1992 [14] tend to favour this analysis method for all structural types, including irregular structures.

WHY ISN'T EVERYONE USING NDP?

Commonly cited reasons for using conventional analysis techniques rather than NDP include:

- Relative computational expense of the procedure,
- Need for more detailed input including appropriate hysteresis rules and appropriately scaled acceleration records,
- Lack of readily available computer software.

In most cases these excuses are no longer a valid justification of the lack of adoption of the NDP in the earthquake engineering profession. With the ever increasing speed of computers, for example, the issue with computational cost is ever diminishing. Kelly [4] presents the example of a 37 storey concrete frame building in Auckland, designed in 1985 using the NDP. At that time, it took 30 hours to analyse the building on a Prime SuperMini Computer. Today the analysis takes less than 1 minute on a desktop computer.

Once the model has been defined, the NDP is a relatively simple procedure to implement and extract results from. Recent developments in performance based design including FEMA 356 [3] tend to push the NSP rather than the NDP, presumably for reasons such as those mentioned above. This seems unusual, given that the level of detail required to define the analysis model for these two procedures is almost identical, and in practice, the NSP is more cumbersome to implement, and has at least as much room for error [7].

Concerns with the determination of appropriate earthquake records for NDP are derived more from inadequacies inherent in the conventional approach of using acceleration response spectra to define input loading. Time histories with similar response spectra can result in very different responses when used with the NDP. This indicates that acceleration response spectrums do not contain all the information representing structural response, a problem that is largely ignored in conventional methods. The choice of time history records with appropriate source characteristics for a given site can overcome this perceived disadvantage with NDP.

THE ANALYSIS PARADOX

In addition to the reasons described above, there are two paradoxes preventing the widespread uptake of NDP. The first is that design guidelines, through scaling procedures, tend to discourage the use of NDP by requiring the application of a higher effective load level relative to other procedures. The result is that under guidelines such as FEMA 356 and the Draft New Zealand Loadings Code, Buildings can be shown to have satisfactory performance under LDP or LSP, but unsatisfactory performance under NDP.

As part of the development of the proposed new Joint Australasian Loadings Code [15], the authors were involved in a study which compared the performance of eight prototype buildings under the proposed

LSP and NDP provisions. In the majority of cases, the peak responses occurred in the NDP analyses. Because of this, there would be no incentive under the proposed new code to adopt the more rigorous NDP. This is counterintuitive as the more accurate technique should always provide a more accurate answer, resulting in a more efficient, less conservative design.

The second paradox is that the apparent difficulties in defining the input for an NDP analysis (including appropriate hysteresis functions, and acceleration records) lead to fears regarding the reliability of the technique. However, most of the simplified procedures we have learned to trust and rely on in its place use the NDP as a benchmark for determining any empirical relationships they embody.

An example is the use of a dynamic magnification factor (ω) in the design of multistory reinforced concrete wall buildings under the New Zealand Codes. A recent study by Priestley [13] highlights concerns of inadequacies in this procedure for certain configurations of buildings, and introduces a new simplified method to account for these, which has been benchmarked using NDP.

Clearly, the applicability of simplified methods will always be challenged when an attempt is made to use them with structural configurations other than those considered in the development of the method. The idealized structures that are used for developing these types of procedures inevitably cover only a very narrow range of real structures. Because of this, the NDP is the only completely reliable analysis method.

Simplified methods are still of great value because they can provide insight as to why a certain type of buildings perform in a certain way, highlighting patterns in the complex problem that is seismic response. In the age of performance based design, however, the use of these methods should be restricted to developing the preliminary design solution. NDP is the only universally appropriate method for verifying the performance of the solution.

IRREGULAR STRUCTURES

Despite the obstacles to the use of NDP mentioned above, there are several types of structure that cannot be effectively modeled using a lower level analysis technique. In the context of this paper, the term 'Irregular Structure' is used to describe any structure that cannot be satisfactorily represented by the simplifying assumptions of analysis procedures other than the NDP. Some examples of structural types that fall into this definition are:

1. Rocking structures
2. Base-isolated structures
3. Structures with supplemental damping
4. Pounding buildings
5. Structures with uneven founding levels
6. Vertical irregularity due to large stiffness changes.
7. Buildings with flexible diaphragms
8. Existing buildings without well defined seismic systems.

Case studies of three of these structural types are presented in the following section. In all cases, the NDP was central in the performance based assessment, and design of these buildings. The assessment procedure adopted was based on Kelly [6], using software that has been developed specifically for this task from the ANSRII analysis engine [8]. This typically involves running a suite of 24 time history analyses per assessment: each combination of 3 earthquake records, 2 primary directions and four eccentricities.

CASE STUDIES

Auckland Art Gallery Seismic Assessment

A detailed study of the existing Auckland Art Gallery buildings was commissioned by Auckland City to assess options for upgrading their seismic performance. The complex consists of four buildings constructed between 1887 and 1981. The older buildings are of unreinforced masonry (URM) construction with timber diaphragms. The newer buildings resist lateral loads through reinforced concrete walls, and reinforced concrete diaphragms.

There are several defining characteristics of the seismic performance of this type of structure that can only be satisfactorily represented if nonlinearity and dynamic effects are explicitly modeled. These include:

1. Flexible timber diaphragms with degrading stiffness and strength properties.
2. Non-ductile, nonlinear behaviour of URM (including shear degradation, rocking piers)
3. Interaction between new and existing materials.

If these characteristics are not accounted for, it is very difficult to assess with any confidence, and without undue conservatism, the level of seismic load that the building can sustain at any given performance objective. Because of this, the NDP was used as the core for the performance based assessment.

Figure 1 shows a rendered view of the building model. URM walls were modeled with panel elements incorporating degrading shear strength and stiffness properties based on FEMA 356 [3]. Figure 2 illustrates the adopted relationship. Rocking of slender piers was included by way of gap elements, preloaded with gravity loads. Timber diaphragms were modeled as flexible, with degrading strength and stiffness properties in accordance with the recommendations contained in [9].

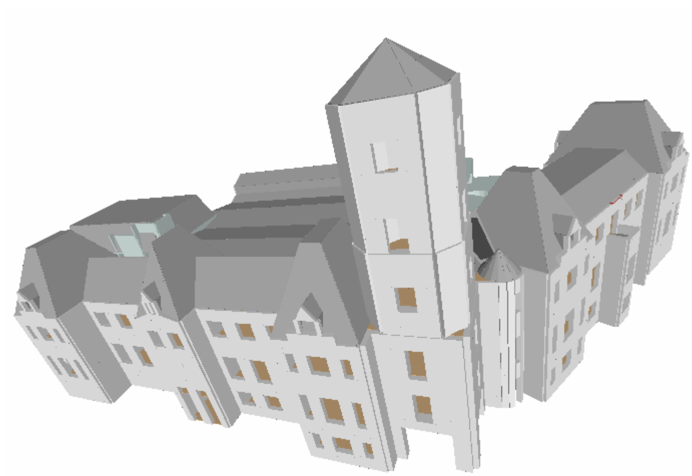

Figure 1: Rendered View of Art Gallery Structural Model

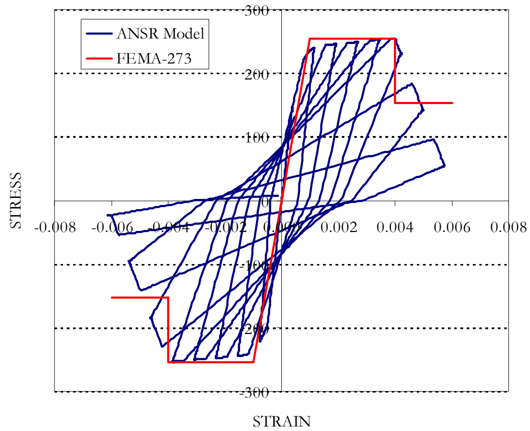

Figure 2: URM shear hysteresis compared to FEMA recommendations

The analysis was able to show that in its existing state, the buildings could sustain approximately 25% of the code load for new buildings without damage. At 50% code level of load, local damage is expected, but the structure remains stable on the whole. At 75% of the design level earthquake, wide-spread collapse is likely, initiating with failure of the floor diaphragms. The in-plane performance of the walls (shear strains and flexural cracking due to rocking) is largely satisfactory at this level of load because the load in these walls is limited by the premature failure of the diaphragms, which effectively acts as a fuse, restricting damage to the upper levels, and preventing high accelerations from occurring in the upper levels.

Because building failure initiated in the diaphragms, the first recommendation in retrofitting the building was to strengthen and secure the diaphragms. To represent the impact of this on the overall performance of the building, the set of analyses were re-run with an assumption of elastic diaphragms. This had the effect of increasing the response in the upper levels in some areas, causing some localized damage. For the most part though, there was shown to be sufficient capacity in the existing structure to provide a satisfactory level of seismic performance. Where the level of damage to URM walls was a collapse issue, a retrofit solution incorporating Fibre Reinforced Polymers (FRPs) was recommended. The NDP is ideal for assessing the effectiveness of this type of solution because an additional FRP element can be added in parallel to the deficient URM elements, and the analysis can be rerun to assess the performance of the retrofitted building. This process is discussed in more detail in Chambers [2].

Additional Levels to Existing Carpark Buildings

A number of projects involving the addition of levels to existing buildings have been constructed recently in Auckland, New Zealand. It can often be relatively straightforward to demonstrate that the existing structures and foundations have sufficient reserve capacity to take the additional gravity loads due to such a development. However, the seismic aspects of the problem are often more difficult to asses.

Two examples of buildings that have recently been developed in this way are the Downtown and Victoria St Carpark buildings. Both are of reinforced concrete construction with lateral load resistance provided by reinforced concrete walls and frames.

In order to minimize the additional gravity loads on the existing structure and maximize the number of additional levels that can be added, it is desirable to adopt a light weight structural form for the new levels. It is also desirable to make full use of ductility in the new system in order to minimize seismic reactions imposed on the existing structure below. This resulting vertical irregularity can in some situations mean the upper structure amplifys the response of the structure below. In other cases it can act as a tuned mass damper, reducing its response. These factors, in combination with the desire to accurately assess an existing structure, compromise the ability of conventional analysis methods to represent the problem.

This difficulty was observed when the intention of adding floors to the Downtown Carpark building was initially met by a structural solution developed using the LSP. The engineers found that the problem pushed the boundaries of what this procedure and the conventional codified design process could cope with, so they opted for a conservative interpretation. The resulting solution largely ignored the inherent seismic capacity of the existing structure, and instead provided a new lateral load resisting structure in the existing levels to track the seismic load from the upper structure down to the ground, where new foundations were also provided.

An alternative solution was then developed using the NDP and the performance based design procedures described in Kelly [6]. This solution required practically no strengthening of the existing structure, and no new foundations, at a substantial cost saving over the original scheme.

The same principles were used to develop a solution for proposed additional levels to the Victoria St Carpark (Figure 3). This building comprises of two structurally separate reinforced concrete buildings, built in 1961 and 1985. The owner of the building wished to increase the carparking capacity of the buildings by adding one floor to the 1961 building and four to the 1985 building.

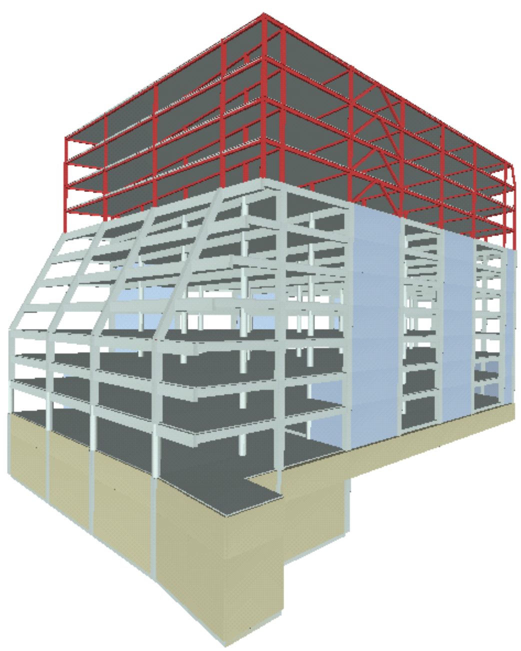

Figure 3: Rendered View of Victoria St Carpark Structural Model

The site slopes steeply from east to west, so the building is seven stories above grade at its west elevation and stories at its east elevation, which added a further degree of complication to the analysis. The lateral load resisting system for the existing building consists of reinforced concrete shear walls, which provide most of the lateral load resistance in the east-west direction. In the north-south direction, reinforced concrete frames provide most of the lateral resistance.

The new structure on top of the existing building was designed to be light weight, with ductile eccentrically braced steel frames above the existing reinforced concrete walls providing its lateral load resistance. Preliminary sizes for the frames were determined using loads based on a LSP analysis. The member sizes were then reduced significantly by using the NDP iteratively until a desired balance between member weight and ductility was met. The NDP was able to show that the four levels could be added without any strengthening required to the structure below.

Tauranga Art Gallery, New Zealand

The Tauranga Art Gallery is an existing 1960's reinforced concrete building which required a seismic upgrade to satisfy the change of use requirements of the New Zealand Building Act. At first assessment, the building appeared to have a reasonably well configured seismic resisting system, with one long wall in the north-south direction and two short perimeter walls in the east-west direction. The building model is shown in Figure 4.

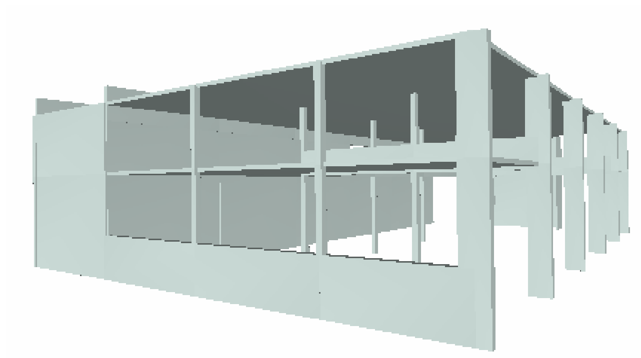

Figure 4: Rendered View of Tauranga Art Gallery Structural Model

Both the flexural and shear reinforcement in the walls was sufficient to take an elastic level of load, however, there was no positive connection between the base of the walls and the foundations, and the foundation beams were inadequately reinforced to transfer any uplift loads to the piles.

If the walls were allowed to rock in their existing configuration, there was insufficient gravity load on the walls to ensure stability. This was confirmed with an NDP analysis of the building. As retrofitting the foundations would have been invasive and relatively costly, a solution was proposed whereby the length of the shorter walls was extended by providing an additional bay of reinforced masonry. This can be seen in Figure 5. With this change in wall geometry, the walls could resist a load equivalent to that determined assuming a structural ductility of 2.0 before becoming the onset of rocking. As confirmed with subsequent NDP analysis, this was sufficient to ensure overall stability of the rocking system.

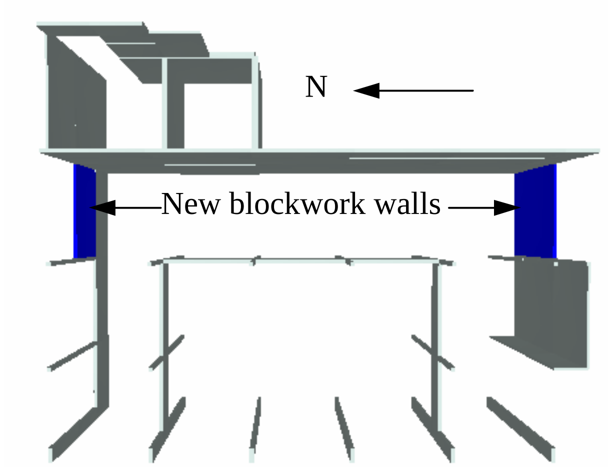

Figure 5: Birds Eye View Showing Extensions to East-West Walls

CONCLUSIONS

Clearly, there are certain types of structure that require the use of the NDP to obtain a reasonable representation of their seismic response. The irregular structure types listed and the case studies discussed above fall into this category. Other analysis methods would either provide dangerously inaccurate assessments of these structures, because they ignore the implications of one or more of the structural characteristics that define structural response, or they would be overly conservative, perhaps limiting the ability to make use of innovative design solutions.

There are many other situations where the NDP can provide a superior assessment over conventional methods. This is because NDP directly addresses effects such as dynamic magnification, building ductility, and P-delta among others, rather than relying on semi-empirical relationships which have been developed by considering a relatively narrow range of structural configurations.

The impediments in the use of the NDP are nolonger sufficient to justify the lack of adoption of this procedure in the industry. The profession needs to change their collective mind set, stop resisting and start embracing what technology now enables us to do.

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