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Testing Methodology for the Dynamic Properties Evaluation of Building Parts and Non-structural Components

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Note

This report is intended for structural engineers, researchers and Standards Committee members.

TESTING METHODOLOGY FOR THE DYNAMIC PROPERTIES EVALUATION OF BUILDING PARTS AND NON-STRUCTURAL COMPONENTS

BRANZ Study Report SR 157 (2006)
Tony Walther

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ABSTRACT

NZS 1170.5:2004 provides clear guidelines for the design of building parts and non-structural components, but without giving designers or manufacturers a means to evaluate the dynamic properties of building parts and non-structural components. An extensive international literature research was undertaken in order to evaluate the applicability of various approaches to the New Zealand testing environment with its 'comparatively' limited test facilities. Feasible alternatives were condensed to a compact and efficient testing methodology. Verification of the testing method was undertaken where applicable.

KEYWORDS

Testing methodology, dynamic properties evaluation, building parts, non-structural components.

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1. OBJECTIVE

This Study Report provides a methodology to evaluate the dynamic properties of building parts and non-structural components.

2. INTRODUCTION

The term 'non-structural components' is ambiguous as it has two main definitions. In research literature, non-structural components are predominantly subdivided into two main categories:

- non-structural building systems such as suspended ceilings, decorative architectural elements, lights, ducts, pipes and non-load-bearing partitions

- building contents such as filing cabinets, hotel room furniture, bookcases, business equipment etc.

This report refers to non-structural components as non-structural building systems only, excluding building contents.

Non-structural components of a building are those systems, parts, elements or components that are not part of the structural gravity or lateral load resisting system but are subjected to the building dynamic actions caused by, for example, an earthquake. Typical examples of non-structural components include architectural partitions, piping systems, ceilings, building contents, mechanical and electrical equipment, and exterior cladding. Sample data shown in Figure 1 from Miranda (2002) illustrates the typical investment in structural framing, non-structural components and building contents in office, hotel and hospital construction. Clearly the investment in non-structural components and building contents is far greater than that for structural components and framing.

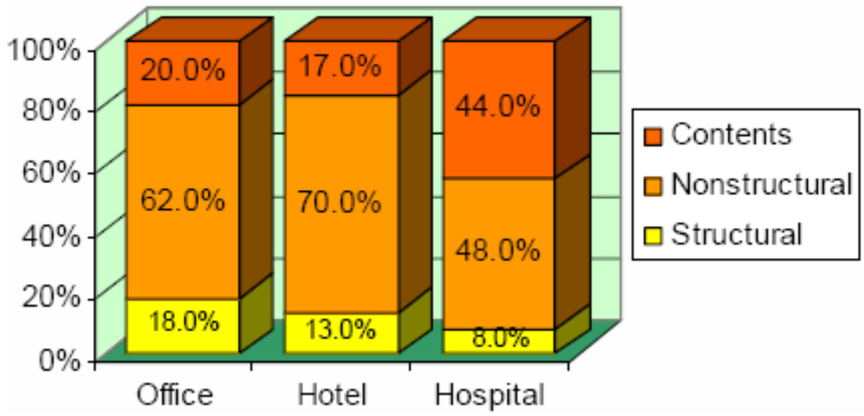

Figure 1. Typical investments in building construction.

The importance of non-structural component issues in seismic design and performance evaluation is now well recognised by researchers as well as practising engineers. The subject received special attention after the San Fernando earthquake in 1971 when it became clear that damage to non-structural components can not only result in major economic loss, but was also life threatening, For example, an evaluation of various hospitals following the San Fernando earthquake revealed that many facilities which were structurally intact were no longer functional because of loss of essential equipment and supplies (Whittaker 2004).

After an earthquake – even a medium-sized one – the structure itself will typically perform as intended, yet the building's non-structural components and contents can be seriously damaged and consequently lead to even bigger losses (e.g. water leakage due to pipe breakage). These indirect economic losses, e.g. stand-down periods due to the unavailability of some plants, pose a great financial risk for many businesses.

This Study Report is broken down into four main parts:

- A literature search for feasible approaches to specific parts was conducted and the findings are given in Section 4. This was deemed as a useful tool for researchers due to the large diversity of non-structural parts.

- The second part (Section 5) describes an approach to a testing methodology for displacement controlled components which extend over one storey only, such as cladding panels, glazing panels etc.

- The third part (Section 6) outlines a methodology for displacement controlled parts extending over more than one storey. Here at BRANZ this approach was found not only easy to set up, but also a good reflection of the in-situ situation to evaluate non-structural components which fall into this category (e.g. ducts, multi-storey glazing, etc).

- Finally, Section 7 provides a methodology to test acceleration controlled components such as lighting, chillers, pumps fans etc. As there are a great variety of items in this category with different dynamic behaviour, examples for the different categories and the possible testing methodologies are also given.

Appendix A outlines the evaluation of the dynamic properties of an exemplar displacement controlled component extending over more than one storey. The author deems this approach, although unconventional, very effective and straightforward, which justifies its consideration when dealing with non-structural components falling in this category.

3. BACKGROUND

In recent years New Zealand has been revising its loadings standards as part of a process of updating based on new information, and jointing with the Australian loadings standards, with the aim of commonality between the two countries. This has been achieved with the majority of loading conditions, but differences in approach between the two countries ultimately led to the decision not to work to the same seismic provisions in both countries. As a result, Australia and New Zealand have produced their own provisions in two separate standards. The new New Zealand standard for seismic loads is NZS 1170.5:2004.

Currently the provisions of NZS 1170.5 draw heavily on the outcomes of BRANZ *Study Report 124*: *Seismic Response of Building Parts and Non-Structural Components*. The structural design of the wide range of parts and components required in a typical building, and their fixings and seismic restraints, requires detailed knowledge of their dynamic characteristics (in particular the fundamental period and ductility including the connection to the main structure). This knowledge is generally outside the expertise of the structural engineering designer of the main building, and for this reason a table of suggested ductilities and periods of parts is included in the commentary of NZS 1170.5. This table is a more comprehensive version of a similar one from NZS 4203, and it draws quite heavily on the National Earthquake Hazard Reduction Program Provisions (NEHRP) which form the basis of several codes and standards in the USA. The technical basis of this data is not understood in the New Zealand context and in many instances the values given might need verification. It is important that the New Zealand engineering community are able to use this data with confidence, and to this end Building Research (BR) agreed to fund a project to set in place procedures to verify and confirm this data for application in this country. The objective of this project is to develop an evaluation method to assess the dynamic characteristics of building parts and determine the fundamental period and ductility of a representative range of parts using that procedure, thus providing the basis for technical input into NZS 1170.5:2004.

In common with other standards worldwide (NEHRP 2003), (NBC 1995), (ASCE 1998) and (ICC 2000), NZS 1170.5 adopted the conventional force-based procedure to determine earthquake design actions on parts. There was some concern among researchers that this approach may not be a good predictor of damage to building parts. The anomaly may be due to high floor acceleration pulses of very short duration and very small displacements caused by building response in the higher modes. Such observed behaviour has been encountered in elastic and inelastic analyses, but does not necessarily result in actual damage.

In contrast to NZS 1170.5, some other standards calculate a force coefficient for the part by means of a multi-factor equation. Generally such equations contain terms quantifying the maximum ground acceleration for the site, the response of the building (depending on period), the response of the part (depending on flexibility or ductility), and a risk factor for the part. To complete the Newtonian analogy, the coefficient (which effectively is acceleration) is then multiplied by the operating weight of the part (Shelton 2004). NZS 1170.5 has adopted a similar approach to the other standards, after consideration of the dynamic response of a series of model buildings to a range of earthquake input motions.

There was a perception that the parts provisions of the old loading standard, NZS 4203 (SNZ 1992), was difficult to apply, particularly since they required detailed information from the seismic design of each specific building. This is a major impediment to the designer or manufacturer of the ‘off-the-shelf’ items that account for a significant portion of parts and components that are installed in new buildings. Also, the treatment of floor accelerations where the building has been designed for over-strength is not clear. The default value of μ = 1.0 used in the equation of floor acceleration to account for over-strength, which is almost universally used by designers, can be shown (Kelly 2001) to result in an over-estimation of floor accelerations by a factor of up to 3. In the new provisions of NZS 1170.5:2004, the designer looks at the parts and components response issue separately from the influence of the building’s response on the part. Also the risk and consequences on the building and on the parts are assessed separately.

The response to the issues raised above is a simple multi-factor equation to determine the horizontal force on the part (see equation 1):

$$F_{ph} = C_p(T_p)C_{ph}R_pW_p \leq 3.6W_p \qquad (1)$$

where

$C_p(T_p)$ = the horizontal design coefficient of the part

C_{ph} = the part horizontal response factor

R_p = the part risk factor

W_p = the weight of the part

The horizontal design coefficient of the part is determined from equation 2:

$$C_p(T_p) = C(0)C_{Hi}C_i(T_p) \tag{2}$$

where

$C(0)$ = the site hazard coefficient for T=0

C_{Hi} = the floor hazard coefficient for level i

T_p = the period of the part

$C_i(T_p)$ = the part spectral shape factor at level i

It is necessary to have a suitable reference point for the calculation of the forces on a part. The maximum value of the input ground motion record (effectively the peak ground acceleration) is one possible parameter, and was used by Rodriguez et al (2000) in a study of floor accelerations. However, this is a variable quantity depending on the record chosen as input to a time history analysis. A well defined value, readily available to the designer, is the level of earthquake hazard at the site, defined by NZS 1170.5:2004 as the elastic site hazard coefficient at zero period, *C(0),* for the appropriate return period.

This is determined from equation 3 in Section 3 of NZS 1170.5:2004:

$$C(T) = C_h(T)\, Z\, R\, N(T,D), \tag{3}$$

where

Ch(T) is the spectral shape factor ($C_h(0)$ for zero period, T=0)

Z is the hazard factor

R is the return period factor *Rs* or *Ru* for the appropriate limit state

N (*T, D*) is the near-fault factor (equal to 1.0 for zero period).

Ultimate limit state guidelines for displacement controlled parts are given by Clause 7.5.1 of NZS 1170.5 which stipulates a maximum inter-storey deflection of 2.5% of the corresponding storey. However, most displacement controlled parts are primarily limited by their service limit states. There are no definite limits given for serviceability apart from Table C8.2 of

NZ 1170.5 – Commentary which provides "Suggested ductility and deformation limits for parts".

NZS 1170.5 seems to be directed in a fashion that the design of the main structure has to accommodate for the parts to stay within its suggested ductility and deformation limits stipulated in Table C8.2 of NZ 1170.5 – Commentary. However, in many cases this might not be a practical approach and it might be equally justified to say that either appropriate parts must be chosen in the first place or amended to ensure the parts stay within their set limits. In this case the building design would be irrespective of the parts, which is certainly preferred from a designer's point of view, and in some cases might even be the only feasible option.

The testing methodology outlined in this Study Report provides the input for displacement and acceleration controlled parts. For acceleration controlled parts, this input is provided in terms of μ and T, to determine the appropriate part Horizontal Response Factor, C_{ph}, and the Part Spectral Shape Factor, $C_i(T_p)$, required in Section 8 of NZ 1170.5, respectively.

4. EXAMPLES OF THE PERFORMANCE OF SPECIFIC PARTS DURING PAST EARTHQUAKES

A more complete description can be found elsewhere (Filiatrault et al 2001, Reitherman and Sabol 1995).

4.1 Performance of elevator and escalator systems

Elevators are one of the most often used plants in building structures nowadays. As a result, they are amongst the most important mechanical systems and are quite susceptible to earthquake-induced damage. With respect to the immediate-occupancy performance level, elevators are vulnerable to service disruptions. Suarez and Singh (2000) described the main components of an elevator system. Consequently they summarised the observed damage as follows:

- damage to guide rail anchors
- bent guide rails
- counterweights dislodging from their guardrails
- loose counterweights impacting passenger cars
- control panels tipped or moved
- traction machines shaken loose from their mountings
- motor-generator sets shifted across machine room floor
- ropes damaged by projections or protuberances in the hoist ways
- suspension ropes jumped from drive
- seismic switches failed to trigger.

Past earthquakes, on the other hand, did not significantly affect escalator systems, until the 1994 Northridge earthquake in California and the 1995 Kobe earthquake in Japan where damage to many escalators was observed. The fact that both earthquakes occurred early in the morning contributed to the lack of injuries to escalator passengers.

4.2 Performance of mechanical, electrical and appliance equipment

Any component with mass (in particular large, tall and/or narrow equipment) which is not adequately anchored is susceptible to slide or overturn during an earthquake. The disengagement can cause damage to the equipment itself or to its connections. Mechanical or electrical equipment mounted on vertical vibration isolators can be particularly vulnerable to being shaken off their isolated supports. In particular, suspended equipment swaying during and after an earthquake pose a high risk to other components, and can also be life threatening. Unanchored water heaters may slide and overturn, which may result in broken water and gas lines; the latter representing a significant fire hazard.

Gates and McGavin (1998) looked at the performance of different anchor options. It was found that during the 1994 Northridge earthquake, mechanical and electrical equipment that was rigidly bolted or anchored to the main structural system performed well, provided that the anchors and supports were designed for code-prescribed loads. On the other hand, equipment mounted on vibration isolation systems (such as rubber or springs) performed poorly. This is mainly due to the unrestrained large displacements that were induced by the ground shaking, as well as amplified inertia forces that caused failure of the anchors. Vibration isolation systems are usually designed by mechanical engineers for reducing occupant discomfort under the machine-induced vibrations, and are then simply treated as flexibly mounted elements when computing the seismic forces. These systems had very large dynamic amplification responses that may have exceeded the amplification factors predicted by codes. This highlights also the need for more coordinated efforts among the various specialties involved in the design and installation of non-structural components.

Approximately 2500 water heaters were damaged during the 1994 Northridge earthquake (Mroz and Soong 1997). In past California earthquakes, water heaters were a major source of gas leaks, posing an important post-earthquake fire hazard. The number of damaged water heaters equipped with 'some kind of restraints' was similar to the number of those without any restraints. This is an indicator that non-engineered restraints were non-effective in protecting water heaters.

4.3 Performance of emergency power systems

The failure of emergency power systems during an earthquake can be particularly disruptive, since these systems are designed to be activated in the event of an emergency. Emergency power systems include heavy components such as batteries, motor generators, fuel tanks, transformers, switchgear and control panels that are frequently stored in racks.

Reitherman and Sabol (1995) found that the loss of off-site electric power during the 1994 Northridge earthquake put the emergency power supply systems to the test, especially for essential operations. The power outage affected over two million people in the Los Angeles area. Merz and Eli (1997) summarised the observations made below during the performance of emergency power systems after surveying a series of electric power facilities, industrial facilities, power plants, and lifelines after this earthquake:

- emergency generators directly anchored or engineered isolators with seismic restraints performed well

- a transfer switch from normal off-site power to emergency power did not function

- a pumping system transferring fuel from a storage tank to a day tank was non-operational because it was not powered by an emergency power system

- failure of a switch from an empty fuel tank to an auxiliary tank caused another emergency generator to be non-functional

- electric shorting in electrical enclosures due to water leaks from domestic water and fire sprinklers caused the shut-down of certain power systems.

4.4 Performance of hazardous material storage systems

Special attention should be paid to the failure of hazardous supply lines and the improper operation of seismically activated shut-off valves as they possess a life threatening potential following an earthquake. For example, toppling of laboratory chemicals should be prevented during seismic shaking. In general, tall vertical tanks used for storing fluids are susceptible to overturning due to the high inertia under seismic loading.

4.5 Performance of ceiling systems

It is unclear whether ceiling systems are more displacement or acceleration controlled. The ceiling grid with its tiles is confined by the surrounding walls, which indicates that they might be displacement controlled and hence suffer under relative displacement of the confining walls. However, the suspension grid holding the tiles is suspended from the concrete slab above via steel wires. This, in turn, implies that grid and tiles interact with the confining walls independently. Film footage from the Kobe 1995 earthquake indicates that the later assumption is a more realistic and likely one. In this footage the ceiling system seems to act independently. The first tile dislodged from the grid and subsequently fell to the ground. This, in turn, allowed more space for the other tiles to move and set off a chain reaction, with many tiles falling to the ground, and finally the whole ceiling grid system collapsing.

Filiatrault et al (2001) observed in the 1994 Northridge earthquake that unbraced suspended ceilings can swing independently of the supporting floor and induce damage, particularly at the perimeters of ceilings. Lay-in ceilings are predominantly vulnerable to the relative displacement of the supporting grid members. During this earthquake, millions of square metres of ceiling tiles were dislodged along with lighting fixtures and air vent ducts (Gates and McGavin 1998). The effect of the fire sprinkler systems that penetrate the ceiling tiles to expose the sprinkler head caused irreparable damage. Code changes released in the USA require spacing between the sprinkler head and the ceiling tile to accommodate the differential movements during the seismic loading. Similarly, no spacing is typically provided to accommodate differential movement between the ceiling grid and the perimeter walls. This also contributed to the extensive damage to ceiling systems during the 1994 Northridge earthquake.

4.6 Performance of lighting fixtures

Fluorescent lighting fixtures that are supported by a suspended ceiling grid can lose their vertical support when the suspended ceiling sways and distorts under ground motion shaking. The splices of electrical wires used to support pendant-mounted lighting fixtures can pull apart, causing the fixtures to fall. Lighting fixtures can also swing and impact adjacent objects, often causing the fixtures to fall or fail.

Failure of light fixtures was one of the three most frequent kinds of non-structural damage suffered by school buildings as the result of the 1994 Northridge earthquake. A new type of lighting fixture damage that was observed during this earthquake was the fall from high-intensity discharge gas vapour lights (Reitherman and Sabol 1995).

5. DISPLACEMENT CONTROLLED COMPONENTS EXTENDING OVER ONE STOREY ONLY

5.1 General

A racking test, similar to the P21 test used in New Zealand to evaluate bracing units (Cooney and Collins 1987, King and Lim 1991), could be used to evaluate the characteristic performance and behaviour of components which are primarily controlled by the application of seismic-induced displacements. In the laboratory, these seismic actions are replicated by slow cyclic application of loads (or deformations) whose history (in terms of the applied load or of a deformation caused by the applied load) follows a pre-determined pattern. This testing regime is not relevant to components whose behaviour is significantly affected by the dynamic response of the component, or are velocity sensitive. This includes components whose behaviour is sensitive to strain rate effects. A more complete description of this section, including the approach to capture and describe the failure modes with fragility curves, can be found in ATC-58 (2004).

5.1.1 Examples for displacement controlled components extending over one storey only

Non-structural components (for which the methodology outlined in this chapter is suitable) include cladding panels, glazing panels and similar components. These are, generally speaking, architectural elements. This section excludes components that extend over more than one storey like pipes, ducts, multi-storey glazing etc, which are covered in Section 6.

5.2 Purpose and application of the methodology for non-structural components

5.2.1 Scope of testing program

In the context of performance assessment, the scope of a methodology is to provide data for the estimation of direct losses (repair or replacement costs, fatalities) or to show compliance with specific functional criteria such as the need to declare the component unfit for a specified function. For both aspects, the scope of a testing procedure is as follows.

5.2.1.1 Identification of relevant damage states

DS's need to be well defined, clearly discernible, and associated with effects whose costs or consequences can be quantified. Such an effect could be the application of a specific repair technique, or the need for replacement of the component, or the creation of a life threatening condition (for example, rapid depressurisation of a pressure containing vessel). It could also be the need to declare the component incapable of fulfilling its function.

For example, Bersofsky (Bersofsky 2004) investigated damage states (DS's) which included: paint cracking, screw head 'popping', drywall fracturing and buckling of studs. Three DS's were identified during testing: Slight (DS1), Moderate (DS2), and Severe (DS3). DS1 was characterised by superficial damage which could be repaired with plaster, tape and paint alone. During testing, the first observed paint crack indicated commencement of DS1. DS2 was characterised by damage to the gypsum panels which could not be repaired with plaster and tape alone, but required replacement of panel sections. DS3 marked the point at which the wall's steel framing was compromised beyond repair. This level of damage required replacement of sections of the entire wall in order for it to be properly repaired.

5.2.1.2 Quantification of damage states

Criteria for DS's are not uniquely defined in most cases, and the decision when a specific DS is attained requires judgement, based mostly on experience. This uncertainty can be reduced by the development of well defined criteria for DS's and by the employment of experts capable of exercising good judgement. For a single test specimen, this uncertainty can be estimated by using a sufficiently large number of experts (experts making the decision on DS's may not have to be present during the test if thorough and complete documentation is provided through measurements, photos, videos and other means of visual observation.). This uncertainty can be reduced by the employment of an instrumentation system that permits the measurement of physical parameters on which the DS's depend (e.g. crack width in a partition).

5.2.1.3 Testing

Testing of the component must be in accordance with a well defined test plan (see Section 5.3) and a loading regime (see Section 5.4). This implies the testing of a sufficiently large number of replicates to permit the quantification of probability functions or, with an appropriate assumption of the type of distribution, the quantification of a central value and a measure of dispersion. It should be recognised that a large number of replicates is theoretically the best, but might not be practical in some cases due to the costs associated or the uniqueness of a part or its system.

5.2.2 Applicable types of non-structural components

This test procedure is intended to be applicable to non-structural components whose behaviour is deformation controlled in one direction. If in-plane and out-of-plane actions are both deemed important, the special considerations discussed in Section 5.4.4 must be incorporated. This methodology is specifically for the testing of non-structural components in which seismic actions are simulated by cycles whose history, in terms of the applied load or of a deformation caused by the applied load, follows a pre-determined pattern. The test procedure must not be applied to components whose behaviour is significantly affected by the dynamic response of the component, or are velocity sensitive. This includes components whose behaviour is sensitive to strain rate effects.

5.3 Test plan

5.3.1 Test specimen

The component whose performance is to be evaluated needs to be isolated from its in-situ surrounding so that it can be tested in a laboratory environment, but still maintain its relation to in-situ conditions. This is perhaps the most critical aspect of a component testing program, as it requires isolation of a component, careful simulation of boundary conditions, and realistic simulation of seismic effects.

5.3.2 Number of test specimens

Uncertainties associated with variability in material properties and construction methods and details can be evaluated only by testing of multi-specimens (unless analytical means can be employed to estimate the uncertainties from material and sub-component tests). However, in some cases the multi-specimen approach might not be economically feasible, as outlined in Section 5.2.1.3. In this case multi-evaluators involved in the testing, especially the observation of the actual test, might be an option for a better record of DS's and their onset. However, the multi-evaluator approach can never provide a better record of the uncertainties with variability in material properties and construction methods and details. Only more test results of specimen replicates can do this.

Thus, a testing program can range from a single-specimen-single-evaluator program to a multi-specimen-multi-evaluator program. A multi-evaluator program is desirable if the DS's interpretation requires much expert judgement, and in cases where it is not feasible to test more specimens. A single-specimen-single-evaluator program is only recommended if the DS's can be clearly identified, and if the material/construction uncertainties are known to be small compared to the other uncertainties.

5.3.3 Simulation of loading and boundary conditions

It is crucial that the testing plan contains a clear and well documented plan for the simulation of all boundary and initial conditions that may significantly affect any of the DS's of interest. This implies that all important DS's should be identified prior to the test, and the sources of damage should be pin-pointed. In-situ boundary conditions that contribute to the initiation and propagation of damage must be properly simulated (i.e. anchorages to structural or other non-structural components as well as imposed force or deformation patterns that are caused by deformations in the elements surrounding the non-structural component to be tested). Hence, for this purpose, the execution of a preliminary monotonic load test of an additional specimen is strongly recommended.

Some non-structural component types could extend over more than one storey. In such cases, the testing program should either contain tests of multi-storey high specimens (see Section 6), or proper boundary conditions should be created to simulate attachment to more than one storey. The boundary conditions that are critical include panel-to-panel (adjacent panels, horizontally and vertically) joints, guides (with minimum friction) to restrain panels completely in-plane throughout racking tests, prevention of uplift in wall systems that may uplift when subjected to lateral load at the top, application of horizontally distributed load at the top (this is accomplished by proper attachments to the moving beam), and connectors that attach the component to the moving beams of the test facility.

5.3.4 Data recording

DS's sometimes can be quantified through direct measurements, but in most cases they have to be assessed through visual observations. A comprehensive log must be kept of all important visual observations, and should be supplemented by frequently taken photos and other means of instantaneous or continuous visual documentation (e.g. sketches, videos etc).

5.3.5 Fabrication of test specimen

The test specimens should replicate in-situ conditions so that material properties, standard construction techniques and boundary conditions are properly simulated. Specimens should be full size, wherever possible, in order to minimise size effects.

If it is impossible to test a full-sized specimen, then additional tests will be needed to quantify size effects to the extent that will permit reliable extrapolation from reduced-scale tests to full-size behaviour.

It is important that the specimens are constructed by tradesmen who do similar installation in real buildings to be sure that conventional construction quality is attained. Special attention must be given to the connections mechanisms to simulate realistic in-situ conditions.

The fabrication of the test specimen must be fully documented, with an itemisation of all parts (and other properties) on which the DS's depend.

5.4 Loading histories

5.4.1 General considerations

The following are important considerations that enter into the decision process for developing or selecting a loading history for slow cyclic testing of non-structural components:

- In general, there are several DS's to consider. The options are to quantify all of them using a single specimen or to use separate specimens for each DS. The preference is to use a single specimen, which appears justified unless cumulative damage becomes a dominant issue for low DS's.

- When a single-specimen approach is chosen, it has to be ensured that the full range where DS's can possibly occur is covered during the test.

- A cyclic loading protocol is commonly chosen to ensure that damage is a cumulative process caused by reversing and cyclic action.

- Cumulative damage is mainly influenced by: the number and relative amplitudes of the excursions preceding the one at which the DS is first observed, as well as on the sequence in which the excursions occur; the mean effect (since excursions are not symmetric with respect to the origin); and possibly the 'loading' rate at which the cyclic loading history is applied to the specimen. This must be considered when developing a loading protocol. However, sequence effects, mean effects and loading rate effects cannot be considered systematically in a slow static testing program.

- A single loading history has to be developed which, in part by statistical evaluation of seismic response data and in part by judgement, represents all the cumulative damage effects at all the DS's that are to be quantified in a test. Clearly, this cannot be done in a rigorous sense; hence, judgement is an important part of this effort.

5.4.2 Proposal for loading history

5.4.2.1 Loading amplitude

To meet the considerations discussed the loading history should consist of cycles of step-wise increasing amplitudes. ATC-58 (2004) recommends two cycles per amplitude should be performed. This seems to be a sensible approach, considering that the following aspects predominantly influence the number and speed of cycles:

- true simulation of the influence on the part
- capacity of the testing equipment
- enough time for observations (i.e. recognising and describing the DS's) during the test.

A conceptual diagram of the loading history is shown in Figure 2.

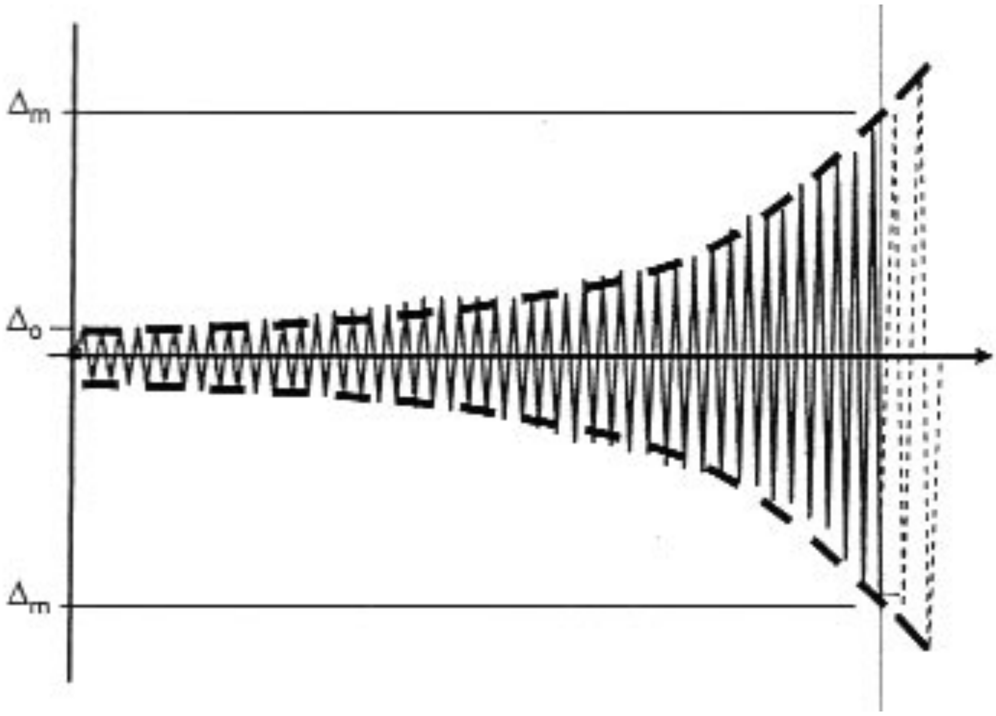

Figure 2. Sketch of loading protocol.

The loading history is defined by:

Δ_o = the smallest amplitude of the loading history (it must be safely smaller than the amplitude at which the lowest DS is first observed i.e. at the lowest DS at least six cycles must have been executed). A value for the smallest amplitude of the loading history (in terms of storey drift index δ/h) of 0.002 is recommended by ATC-58 (2004).

Δ_m = the largest amplitude of the loading history. It is estimated as the value at which the largest damage level occurs. This value is not known prior to the test (although it can be estimated from a monotonic test). If the last DS occurs at a drift smaller than the target value, judgement must be used to assess whether this unpredicted outcome is feasible due to the cumulative damage effect or variability in construction material and installation methods or any other reason. If the last DS has not yet occurred at the target value, the loading history must be continued by using further increments of amplitude. A recommended value for the targeted largest amplitude of the loading history (in terms of storey drift index δ/h) is 0.025 as stipulated for the serviceability limit state in NZS 1170.5:2004.

Whenever possible, the test should be continued beyond Δ_m (even if the last DS has been attained), and should be terminated only when the capabilities of the test set-up have been reached or the test specimen has so severely degraded that nothing additional can be learned about its performance.

a_i = the amplitude of the cycles, as they increase from Δ_o to Δ_m, i.e. the initial amplitude (for the first two cycles) is Δ_o, and the last planned amplitude is Δ_m (or close to it).

5.4.2.2 Loading speed

A maximum stroke travel of 2–5 mm per second is recommended by Thurston (2004). This seems appropriate, considering that the aspects listed in Section 5.4.2.1 apply here too.

5.4.3 Alternative loading history

It must be mentioned that in general the proposed loading history of Section 5.4.2 is deemed sufficient. In special cases, however, the alternative loading history outlined below can be a useful alternative way, which in fact is an adjusted loading history described in Section 5.4.2.

This alternative uses a separate specimen for performance evaluation at each damage level, plus an additional specimen on which to perform an initial monotonic test. The purpose of the monotonic test is to estimate the deformation amplitude at which the targeted performance levels are expected to occur. Once the drifts associated with the individual DS's have been estimated, separate specimens will be subjected to cyclic histories that are associated with the targeted damage levels. The advantage is that each specimen could be subjected to a full floor displacement history before the targeted damage level occurs, but the accumulated damage would not be carried over to the performance evaluation at the next larger damage level. The disadvantage is the need to test several specimens. In this context it needs to be evaluated where the pay-off is bigger: testing several specimens to get statistical data at various damage levels and recognising that cumulative damage may not be 'best' represented; or testing several specimens to get single data points but maybe having more consistent representation of cumulative damage.

5.4.4 Biaxial loading history

Bi-directional testing should be executed if so deemed necessary. The loading history for bi-directional testing must follow the orbital pattern shown in Figure 3. This loading history has only one cycle per amplitude. Then the amplitude is increased to the next highest by i.n – i.n-1. Whether this increment is constant or follows a function is contingent on the circumstances of mainly the part and the in-situ conditions which try to be simulated. This decision lies with the tester.

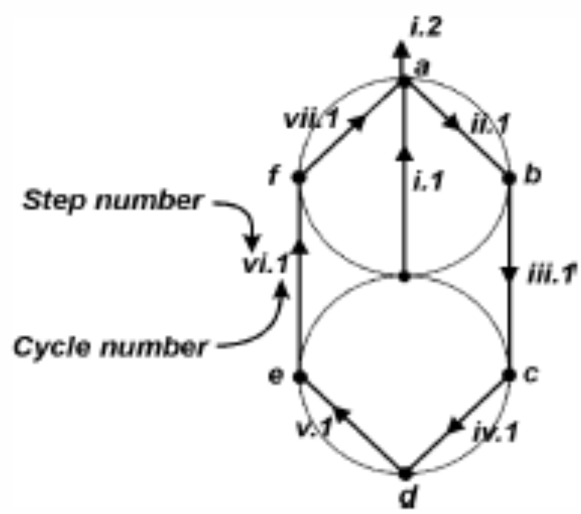

Figure 3. Horizontal plane displacement orbit for performing drift-controlled bi-directional loading tests.

5.5 Standard compliance

Finally, the achieved deformation testing results have to be compared to the ultimate and service limit state thresholds stipulated in NZS 1170.5. For serviceability, the displacements present at the onset of the first DS have to be at least lower or equal to the deformations from Table C8.2 of NZ 1170.5 – Commentary which provide "Suggested ductility and deformation limits for parts". The threshold for ultimate limit state is a deflection value of 2.5% inter-storey drift. It is predicted that in most cases serviceability limits will be governing.

6. DISPLACEMENT CONTROLLED COMPONENTS EXTENDING OVER MORE THAN ONE STOREY

6.1 General

In some instances, the simulation of an inter-storey drift over a single storey is not sufficient to stretch a particular part or component to a certain target DS and the ultimate limit state is sometimes impossible to achieve. For example ducting, piping or glazing extending over more than one storey are preferably tested on a two-storey high specimen connected to three floors. Inter-storey drift can be simulated by displacement of the attachment simulating the middle floor of the building structure. This allows isolation of the tested item and realistic and authentic boundary conditions. The methodology outlined in this section defines the test set-up to examine the properties of the part (i.e. strength and stiffness), but especially the deflection of onsets at discrete DS's. At the same time, it gives the tester the option to add or remove additional restraints (e.g. duct clamps or bracings).

An example test to verify the methodology for displacement controlled components extending over more than one storey was conducted. The responses of a number of pipes from a multi-storey plumbing shaft were simulated. This test represents a typical example of application that fall into this category. For the full test description see Appendix A.

6.1.1 Examples for displacement controlled components extending over more than one storey

The methodology described in this section pertains to non-structural components which are affected by the inter-storey drift of more than one storey. Pipes, ducts and curtain wall glazing are typical examples for this category.

6.2 Test plan

The plan for the test should consist of the following steps:

- pre-test material inspection
- fabrication of test specimen
- definition and documentation of functional performance and anticipated failure modes
- cycle intensity
- initial performance evaluation test
- failure tests.

6.2.1 Pre-test material inspection

Upon arrival at the test facility, the material (e.g. pipes, ducts, etc) must be visually examined and results documented by the testing laboratory to verify that no damage (i.e. cracks, hair fractures, kinks etc) has occurred during shipping and handling. Test descriptions and results must be documented.

6.2.1.1 Variability in material properties and construction methods and details

Procedure should be as outlined in Section 5.3.2.

6.2.2 Fabrication of test specimen

Procedure should be as outlined in Section 5.3.5.

6.2.3 Definition and documentation of functional performance and anticipated damage states

Prior to testing, functional performance and anticipated DS's must be defined for the envisaged test specimen. Clear definitions of functional performance and anticipated DS's are necessary since a level of performance for one test specimen (e.g. pressure loss in the pipe system) may correspond to a DS for another test specimen (e.g. hair fracture). Once these functional and performance levels are defined, they must be documented as they occur in the test specimen during the test program.

6.2.4 Cycle intensity

The input displacement used to define cycle intensity should be the greater of ultimate or serviceability limit state inter-storey deflection. This peak excursion is the targeted largest amplitude of the loading history. For instance, in the exemplar pipe test (outlined in Appendix A) NZS 1170.5:2004 was used to stipulate the targeted largest amplitude of the loading history in the form of a limit state inter-storey deflection. In Paragraph 7.5.1, NZS 1170.5:2004 calls up a maximum ultimate limit state inter-storey deflection of 2.5%.

Whenever possible, the test should be continued beyond the targeted largest amplitude of the loading history (even if the last DS has been attained) and should be terminated only when the capabilities of the test set-up have been reached or the test specimen has so severely degraded that nothing additional can be learned about its performance.

6.2.5 Initial performance evaluation test

The evaluation testing is basically the first step in the testing regime to see whether the envisaged sequence of events was correct, or whether the prediction needed re-consideration based on the results from the evaluation test.

It is recommended that the seismic performance of the test specimen, based on the functional performance and the anticipated DS's stipulated in Section 6.2.3, should be evaluated under simulated inter-storey drifts of increasing displacements.

It is anticipated that at the conclusion of the performance evaluation tests, the test specimen would have responded beyond its proportionality limit, but should not have fully developed its anticipated DS's.

6.2.6 Failure tests

The failure tests mark the second and final step in the testing regime. In cases where the evaluation test yielded the predicted results, a seamless transition to failure testing is possible.

Higher displacements than used in the performance evaluation tests must be used to induce incipient failure at the test specimen. Multiple failure tests may be conducted if the test specimen is composed of various sub-systems that reached incipient failure separately at various intensities. Failure test description and results must be documented for each intensity level. Special care must be taken to ensure the safety of the test personnel and to avoid damage to the tested specimen.

The alternative is to use different specimens for each DS, and apply to each specimen a statistically representative deformation history in which the maximum excursion is deemed to be the excursion at which the DS is attained for the first time. In order to assist in estimating this target maximum excursion for each DS, it is most helpful to perform first a monotonic test on a separate specimen. Thus if n DS's are to be evaluated, the testing program would require n+1 test specimens to provide one piece of data for each DS.

7. ACCELERATION CONTROLLED COMPONENTS

In general, there are two options:

- computer simulation
- testing.

A theoretical approach is possible through finite element simulation or other elasto-PP time history computer modelling (e.g. Ruamoko, ETABS). Time history modelling of non-structural components requires sound understanding of the boundary conditions and constraints, suitable input records, and especially knowledge of the properties of the non-structural component. This last point should be highlighted as it brings a difficult task upon the structural engineer who, in order to determine the dynamic characteristics, has to know how the equipment operates and what the likely order of failure will be. Thus, computer modelling might be applicable in rare instances due to the required interdisciplinary knowledge (structural/electrical engineering) to predict not only the boundary conditions properly, but also to model the electrical plant accurately. For some parts, computer modelling is ruled out due to its complexity. The computer-based alternative is not part of this report (which covers testing and a possible methodology only).

The methodology for assessing non-structural elements that are acceleration controlled is not as straightforward as the displacement controlled components for various reasons.

7.1 General

Damage of non-structural parts in past earthquakes has shown that generally parts that have been installed properly have a far higher probability of performing satisfactory than those with partial, or even no, appropriately engineered fixings (Filiatrault et al 2001, Reitherman and Sabol 1995).

The diversity of acceleration controlled non-structural parts is large. It stretches from mechanical to electrical non-structural components. Thus there is no 'one-model-that-fits-all' testing methodology approach. However, it is possible to give holistic guidelines and recommendations to generic non-structural acceleration controlled parts (see Section 7.2).

7.1.1 Examples of acceleration controlled components

The methodology outlined in this chapter covers mainly *mechanical components* (such as chillers, pumps, fans and cooling towers) on the one hand, and *electrical components* (including motor control centres, transformers, lighting and distribution panels) on the other.

7.2 Holistic guidelines and recommendations

The shake table testing methodology outlined in this section establishes minimum procedures for the experimental quantification of the dynamic properties of architectural, mechanical, electrical and non-structural building components and systems that are permanently attached to a building structure. A more complete description, including fragility curves, can be found in ATC-58 (2004).

Only non-structural components that are mainly sensitive to the dynamic motion (acceleration and velocity) of a single connection level in a building structure are addressed in this section. Components sensitive to the relative motion of several connection levels, such as wall partitions sensitive to inter-storey drifts or piping systems sensitive to the motion of multiple connection points, are not covered in this section (see Section 5 and Section 6, respectively).

The main objectives are:

1. Evaluate the natural period.

2. Evaluate the damping ratio.

3. Evaluate ductility.

This Study Report outlines a methodology to evaluate the natural period and damping ratio of an acceleration controlled part.

7.3 Ductility

NZS 1170.5 stipulates that the ductility of the part must be $\mu_p = 1$ when determining the horizontal part response factor C_{ph}, unless the level of floor acceleration is such as to bring about yielding of the part. The same value of $\mu_p = 1$ applies when determining the vertical part response factor C_{pv}, unless otherwise determined by special studies. When calculating for serviceability limit states, $\mu_p = 1$ must also be applied. On the other hand a value of $\mu_p = 1.25$ must be used when designing for non-ductile connections. Most connections fall into this category. A greater value for μ_p can be used where special studies are employed to verify that the connection can sustain not less than 90% of their design action effects at a displacement greater than twice their yield displacement under reversed cycle loading. However, in most cases the cost of testing is likely to outweigh the cost of stronger fixings, and the quickest and least risk option for designers is to remain with the conservative values for μ_p.

A literature research revealed that there is no ‘one’ recognised way to test or evaluate for ductility. The problem for this lies in the lack of a uniform definition of ductility. The literature research showed that the definition can not only change from employed design system to design system, but also from material to material. The definition to assess ductility tested in a P21 test to evaluate bracing units for NZS 3604, for instance, is very different from the ductility definition which builds the basis to evaluate timber joints. A very common ductility definition for mild steel is given in equation 4:

$$\mu_{Mild\_Steel} = \frac{\Delta_{ultimate}}{\Delta_{yield}} \qquad (4)$$

$\Delta_{ultimate}$ = deflection at failure

Δ_{yield} = yield deflection

However, most other materials and systems do not have the unique stress-strain diagram of mild steel with the distinct yielding point. Thus, in most other design systems and materials a clear definition for a distinct yielding point is a problem area. Hence, a case-to-case problem identification and approach is the preferred solution and will in most instances achieve the most accurate results.

For the outlined reasons this methodology does not provide a generic approach to test for ductility.

7.4 Test plan

The specimen must be subjected to a test plan consisting of the following test steps:

- pre-test inspection and compliance verification
- definition and documentation of functional performance and anticipated failure modes
- system identification tests
- seismic performance evaluation tests
- failure tests.

7.4.1 Pre-test inspection and functional compliance verification

Refer to Section 6.2.1.

7.4.2 Definition and documentation of functional performance and anticipated failure modes and damage states

Refer to Section 6.2.3.

7.4.3 System identification test

System identification must be conducted in order to determine the initial natural period (if possible also the second mode period or higher) of the test specimen, and also the changes of these natural periods throughout the test program due to the changes in DS's. Preferably, single axis system identification tests in each orthogonal direction of the test specimen should be conducted before each of the performance evaluation and failure tests described in the next two sections.

To evaluate the natural period of the test specimen, a minimum of one of the following methods (outlined in Section 7.4.3.1 and Section 7.4.3.2) is recommended to use for each orthogonal direction.

The fundamental damping of the test specimen should be determined based on a minimum of one of the methods for each orthogonal direction described in Section 7.4.3.2 and Section 7.4.3.3.

7.4.3.1 Single-axis acceleration-controlled sinusoidal sweep test

A single-axis acceleration-controlled sinusoidal sweep from 0.25 to 8 Hz must be performed preferably in each orthogonal axis of the test specimen in order to determine its natural frequencies. The sweep rate must be two octaves per minute, or less, to ensure adequate time to establish steady-state response at the test specimen. The peak intensity of the sweep must be limited to 0.1 ±0.05 g. A lower acceleration input level may be used to avoid damage to the test specimen. The natural period must be obtained from the peaks of the acceleration-frequency domain.

In some instances, it is rather simple and does not require much additional work to determine the damping component of the test specimen (thus it is recommended to do so in these cases).

7.4.3.2 Pull tests

Alternatively, the fundamental period and damping ratio of the test specimen can be established by the free-vibration decay measured by the in-line accelerometers as a result of a static pull-back test at the centre of gravity of the test specimen. The intensity of the pull-back force must be small enough to avoid damage to the test specimen.

7.4.3.3 Damping resonance tests

A very low-intensity acceleration-controlled sinusoidal input at the previously identified fundamental period must be used to excite the test specimen in each of its orthogonal configurations. The intensity of the sinusoidal input must be established based on recorded or visual response so that no damage to the test specimen under this resonance condition occurs. Once a steady-state response is established, the input must be suddenly stopped and the free-vibration response decay must be recorded by the in-line test response monitoring sensors. The fundamental damping ratio of the test specimen should then be established by the logarithmic decrement method applied to the free-vibration response decay curves.

7.4.4 Initial performance evaluation test

It is recommended that the seismic performance of the test specimen, based on the functional performance and the anticipated failure modes stipulated in Section 7.4.2, should be evaluated under simulated motions of increasing intensities.

It is anticipated that at the conclusion of the performance evaluation tests, the test specimen would have responded beyond its proportionality limit, but should not have fully developed its anticipated failure modes.

7.4.5 Failure tests

Higher intensities of the simulated motions used in the performance evaluation tests must be used to induce incipient failure at the test specimen. Multiple failure tests may be conducted if the test specimen is composed of various sub-systems that reached incipient failure separately at various intensities. Failure test description and results must be documented for each intensity level. Special care must be taken to ensure the safety of the test personnel and to avoid damage to the tested specimen.

7.5 Shake intensity

The input motion parameter used to define intensity should be the peak spectral acceleration.

7.5.1 System identification tests

It is crucial to choose a shake intensity for the identification tests that is low enough to avoid damage to the test specimen.

7.5.2 Performance evaluation tests

Three different shaking intensities must be used for the evaluation tests. The first intensity level should generate a seismic response of the test specimen not exceeding 50% of its elastic proportionality limit or equivalent. The second intensity level should generate a seismic response of the test specimen approaching its elastic proportionality limit. The third intensity level should generate non-proportional seismic response of the test specimen, but without complete failure. In all cases, a 50% increase in intensity should be the minimum step size between intensity levels.

7.5.3 Failure tests

The shake intensity for the failure tests should induce incipient failure of the test specimen if this has not been done in the last step of the performance evaluation test phase already. If multiple failure tests are conducted, each of the shaking intensities should induce the incipient failure of a particular sub-component or sub-system of the test specimen. The intensity of the failure tests can be estimated by extrapolation from the results of the performance evaluation tests or by other analytical means. A 50% increase in intensity must be the minimum step size between intensity levels.

Whenever possible, the test should be continued beyond the peak spectral acceleration (even if the last DS has been attained) and should be terminated only when the capabilities of the test set-up have been reached or the test specimen has so severely degraded that nothing additional can be learned about its performance.

7.6 Shaking directions

The system identification tests, outlined in Section 7.4.3, must be applied as single axis tests in each orthogonal direction of the test specimen. The performance evaluation (described in Section 7.4.2) and failure tests (Section 7.4.5) are preferably carried out as tri-axial tests with simulated input motions applied simultaneously in all orthogonal axes of the test specimen. Alternatively, bi-axial (horizontal and vertical direction) performance evaluation and failure tests can be used. Also horizontal (bi-axial or uni-axial) performance evaluation and failure testing only is possible. However, this method should be used preferably when it can be demonstrated by analysis or other means that the effect of vertical motions on the seismic response of the test specimen is negligible.

7.7 Data recording

It is recommended that the test data should be acquired in any orthogonal direction of the test specimen with at least 200 Hz. This will ensure that the natural frequency range of any particular part will be found when conducting sinusoidal sweep tests. After the frequency range has been evaluated, another scan with a minimum of 30 readings per cycle must be conducted if the sampling rate was lower during the first test run.

7.8 Input motions

In an earthquake every mechanical, electrical and architectural part is governed by the motion of the floor it is installed on (i.e. the level which provides the attachment for the part). Acceleration of the floors of buildings is a result of the 'magnification' and 'frequency filtering' of the ground motion by the main building structure. It has previously been assumed that this process results in an approximately linear increase of floor acceleration with height up the building (that is the response is first mode dominated). Recent non-linear time history analyses carried out by Shelton (2004) on a representative sample of buildings show that the envelope covering the floor acceleration is linear until a certain point, but from then onwards constant up the building to the top. Hence, a building designer can make the simplification to assume a similar or even same floor acceleration for upper storeys. However, they need to bear in mind that by doing so he exposes his calculations to a certain error which only a full-time history analysis can quantify.

Every building structure is based on a unique design concept, giving the building its ability to dissipate energy which in turn influences the behaviour and response of a building when subjected to earthquake actions.

Compared to the ground motion for a building, which is predominantly governed by the structural design concepts applied, and the composition of the soil the structure is based on, the floor motion of a part has additional variables which further increase the complexity. In most cases the costs required to create a full simulation model of a building structure in order to conduct a time history analysis accurately will not be justifiable considering the value of the part or the consequence of its failure. An economical feasible resolution to this dilemma is finding a part with similar building boundary conditions (i.e. the soil the building is built upon, floor in which part is installed, design of building structure etc). However, considering the sparseness of floor acceleration time spectra records, and the outlined high variety of scenarios, finding a matching one will be rather the exception. Clearly this gives hope that in the future more accelerographs to record floor motions are installed to increase the number of available floor motion records.

It is recommended to evaluate the part first (this evaluation should incorporate at least the following):

- the value of the part
- the importance of the part's operation,
- and the possible loss associated with a certain DS.

Based on the outcome of this evaluation a suitable process to evaluate the input floor acceleration time history must be chosen. The pool of options to consider must include, but not be limited to, the following:

1. A full-time history analysis (non-linear) on the actual building structure must be carried out. Subsequently, the floor acceleration time history record must be extracted from the time history analysis.

2. An existing time history analysis from a similar building, and its associated floor acceleration time history record with similar boundary conditions, can be found and used.

3. An actual record of a floor acceleration time history from an accelerograph installed at a similar floor level and site can be found.

Generally, if the mass of the part (secondary system) is small compared to the building (primary system) an uncoupled analysis can be used. As the mass of the part increases as a fraction of the building mass, there is an increasing need to include the mass and/or stiffness of the part in the building analysis used to derive the input motions for assessment of the part. NZS 1170.5, for instance, asks for a special study to be carried out where:

- the mass of the part is in excess of 20% of the combined mass of the part and the primary structure and its lowest translational period is greater than 0.2 seconds

- the mass of tanks or vessels (including contents) exceeds 10% of the mass of the structure

- the tank is of such size or the design of the support frame is likely to have significant response in its own right.

Where both the part (secondary system) and the building structure (primary system) are combined in one model, option 1 may be the only option available. Such items as the 'existing motions' defined by options 2 and 3 do not account for the mass/stiffness of the part.

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9. APPENDIX A – VERIFICATION TEST OF DISPLACEMENT CONTROLLED COMPONENTS STRETCHED OVER MORE THAN ONE STOREY

9.1 General

An example test to verify the methodology for displacement controlled components stretched over more than one storey was conducted. A pipe ducting in a multi-storey riser shaft was simulated. This was chosen to represent a typical example of an application that falls into this category.

9.2 Test plan

The test plan consisted of the following test steps as per the outlined testing methodology:

- pipe and valve inspection
  (pre-test material inspection)

- installation of pipe system
  (fabrication of test specimen)

- anticipated pipe failure modes
  (definition and documentation of functional performance and anticipated failure modes)

- cycle intensity

- initial pipe system testing
  (initial performance evaluation tests)

- pipe failure test
  (failure tests).

9.2.1 Pipe and valve inspection

Upon arrival at the BRANZ premises in Judgeford, the three different pipes were examined and results documented by the technicians. This ensured that none of the pipes had any visible cracks or kinks suffered from handling or transportation in the delivery process. Since no obvious damage was found, proof testing of the pipes and valves was discarded.

9.2.1.1 Variability in material properties and construction methods and details

Uncertainties associated with the variability in material properties was not an issue considering that pipes are typically mass-produced commodities, usually with stringent in-house quality control systems in place. Also the construction methods (joining of pipes, fixing of fire collars, mounting of bracings etc) and the associated details needed to set up the specimen are very common. Thus testing of multi-specimens, especially considering the high costs associated, seemed unjustified in this case.

A single-specimen-multi-evaluator program approach was chosen as the DS's interpretation required much expert judgement. In addition, DS's were difficult to predict and identify. A replicate of the specimen seemed economically not feasible.

9.2.2 Installation of pipe system

The entire pipe work was done by a professional plumber who predominantly does similar installations in real buildings, especially multi-storey constructions. This ensured that conventional construction quality found out in the field was exercised.

The main aim was to set up the test specimens in a way that it replicated the in-situ conditions so that material properties, standard construction techniques and boundary conditions are properly simulated. It was chosen to build a full-size specimen in order to avoid size-factoring distortions.

The specimen consisted of three different pipes as shown in Figure 4: 100 mm diameter PVC waste pipes, 50 mm diameter polypropylene (PP) water supply and 40 mm copper supply pipes. The PP pipe product used required a special device operated by certified craftsmen to joint two pipes. The device, which was similar to an iron, partly melts the two ends before butting them together (this ensures a very robust and leak-proof connection). A copper pipe version was chosen upon the consulted plumber's suggestion, as according to his experience copper is still used in some cases. However, the trend in the building industry tends to go to the PP pipe version and away from copper pipes in multi-storey buildings.

A manifold (see Figure 5) delivered mains pressure to the PP and copper pipes to simulate the water supply situation in a multi-storey building, and to facilitate the detection of hair fractures or leaks. The PVC pipe was also filled with water in order to assist detecting water leakage but was not pressurised.

Two pipes of each material (PVC, PP and copper) were used. One of each material type extended the 6.8 m of the two storeys without any joints. The other pipe featured two different types of joints: 'T' and 45° joints seemed most common in building practice and hence were chosen (see Figure 6). These were fitted just before and after the timber board simulating the middle floor to maximise the pressure around the joints. PVC pipes were connected with PVC solvent cement, PP pipes with the melting-and-butting method outlined above, and the copper pipes were welded. Also pipe clamps were mounted at distances stipulated in AS/NZS 3500:2003 *Plumbing and Drainage*. However, they were only fixed to the pipes at correct spacing at this stage to be readily available when needed during testing.

Fire collars and fire insulation were installed at the timber boards which simulated the floors as depicted in Figure 7.

The final test set-up is shown in Figure 8.

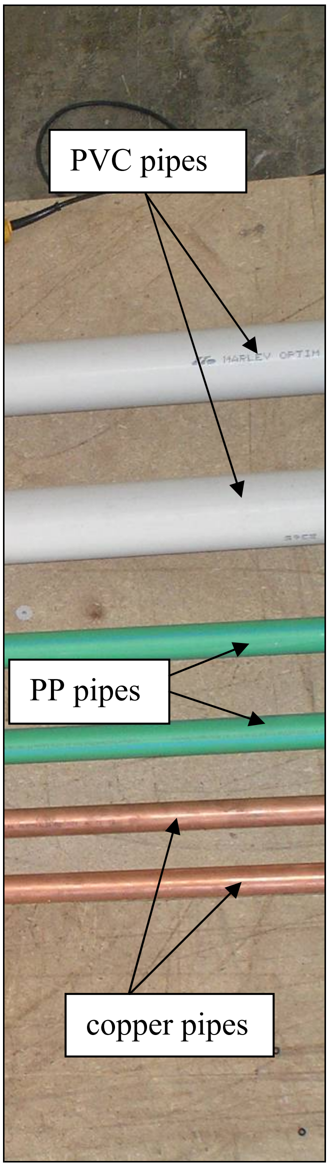

Figure 5. Valves controlling main pressure.

Figure 4. Pipe types.

Figure 6. Pipe 'T' and 45° joints.

Figure 7. Fire collars and fire insulation at timber board simulating floor.

Figure 8. Final test set-up.

9.2.3 Anticipated pipe failure modes

Considering that only one specimen was available, the multi-evaluator approach (see Section 9.2.1.1) was chosen to get as many different views and aspects on possible failure modes and associated DS's. Pressure gauges were attached to the pipes with main pressure to facilitate any resulting failure detection due to gradual pressure loss.

The PVC pipes developed the highest stresses at mid-span due to the maximum bending moment induced at that point. This would apply in particular to the continuous pipe without joints extending the full 6.8 m.

The general opinion was that PP water supply pipes will generate enough flexibility, and that kinking or some hair fracturing is rather unlikely apart from at the connection point.

Taken that the copper pipes are non-ductile, it seemed very likely that kinking and subsequent hair fracturing might occur.

9.2.4 Cycle intensity

The input displacement used to define amplitude was taken as the ultimate limit state inter-storey deflection. NZS 1170.5:2004 was used to stipulate the targeted largest amplitude of the loading history. In Paragraph 7.5.1, NZS 1170.5:2004 calls up a maximum ultimate limit state inter-storey deflection of 2.5% which equals ±85 mm at a storey height of 3.4 m.

9.2.5 Initial pipe system testing

A servo-hydraulic actuator with a stroke capacity of ±300 mm and a maximum 10 tonnes of push/pull force was attached to the board simulating the middle floor (see Figure 9). Cyclic displacement was subsequently applied to this middle floor, while the other two floors (located 3.4 m to each side of the middle floor) were held in place and acted as end restraints.

The test was started with three cycles at a ±20 mm displacement (see Figure 10). Subsequently, displacement increments of ±10 mm were used until a total displacement of ±85 mm was achieved (as shown in Figure 11). 85 mm equals the ultimate limit state inter-storey deflection of 2.5% stipulated in Paragraph 7.5.1 of NZS 1170.5:2004. At this point it was decided to stop the initial pipe system evaluation test as the envisaged sequence of events was correct and the prediction did not therefore need re-consideration.

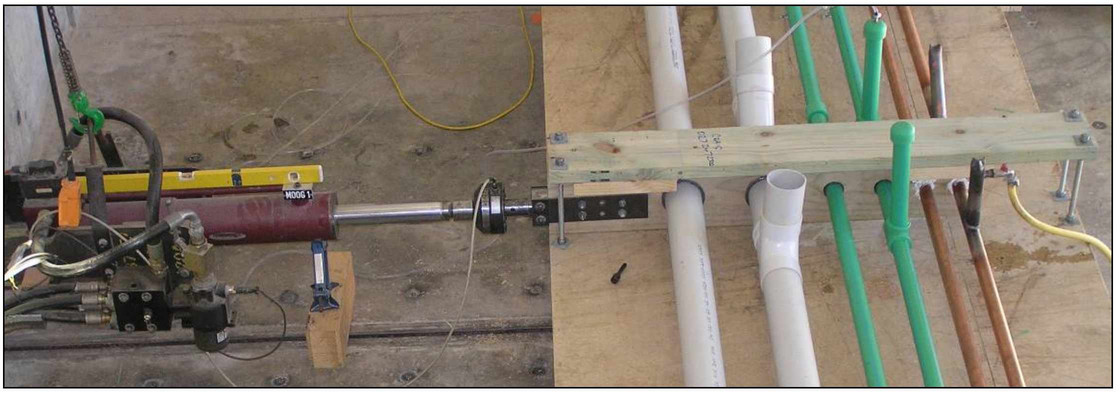

Figure 9. ±300 mm stroke and 10 tonnes actuator used in the test.

Figure 10. Start of test at ±20 mm displacement.

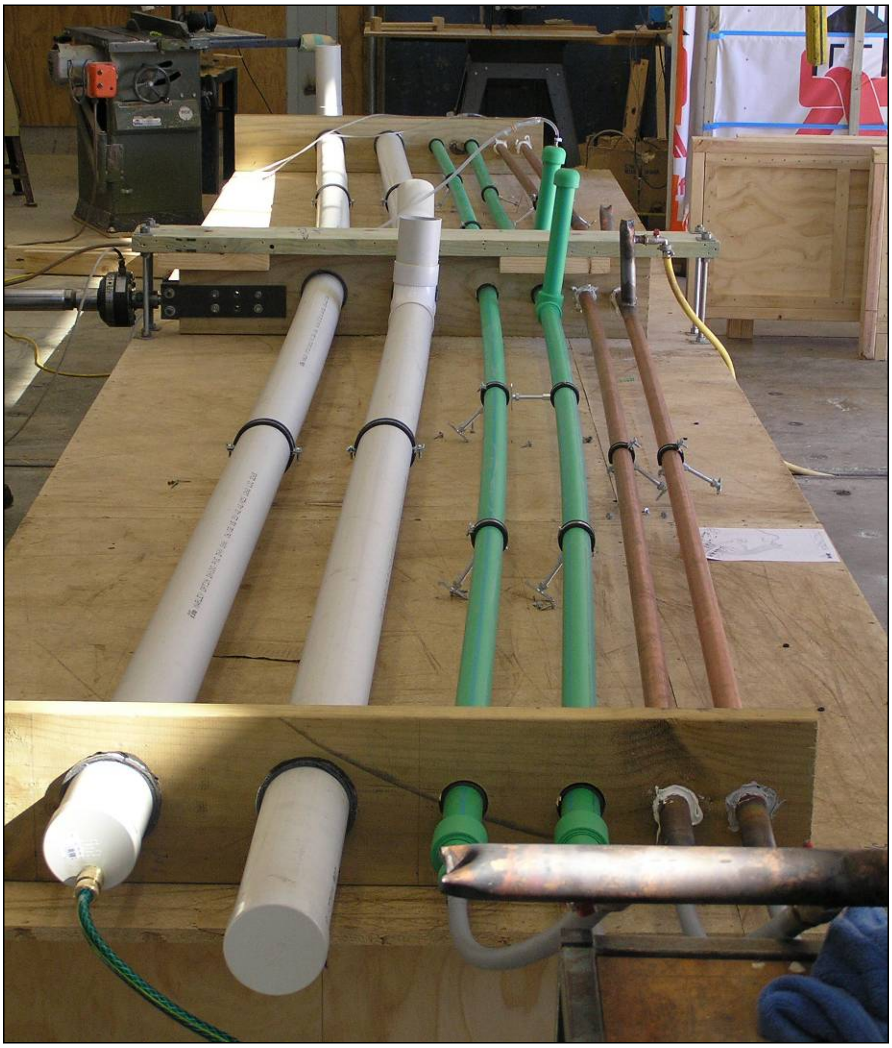

Figure 11. Ultimate limit state inter-storey deflection of ±85 mm.

9.2.6 Pipe failure test

For the failure test, higher displacements than in the performance evaluation tests were used to induce failure at the test specimen. Thus the test was continued, starting at ±85 mm with increments of ±15 mm. At each displacement three cycles were conducted. At a displacement of ±150 mm (see Figure 12), the decision was made to stop and to impose some restraints introducing additional stress.

At this stage the pre-installed pipe clamp fixings, outlined in Section 9.2.2, were fastened to the floor (see Figure 13). The test was re-started at ±40 mm displacement and after every third cycle a displacement increment of ±20 mm was applied. At the ultimate limit state inter-storey deflection limit of ±85 mm no visible damage to the pipes or pressure loss was detectable (as shown in Figure 14). However, the onset of the pipe clamp loosening was observed at this stage (see Figure 15).

At 150 mm (the displacement at which the test without pipe clamps was stopped), significant bending of pipes and the pipe clamps could be seen as shown in Figure 16. However, no irreversible deformation or hair fracture leading to pressure loss occurred. It was noted that some pipe clamps lost part of the hold-down fixings (see Figure 17).

The test was stopped at a displacement of ±270 mm. At this stage, very significant deformations were observed during the test (see Figure 18 and Figure 19). However, only one deformation was irreversible. This deformation was the kinking of a copper pipe as shown in Figure 20. However, no pressure loss was recorded. Also the majority of pipe clamps sustained severe damage (see Figure 21). Nail pull-out or destruction of pipe clamps led in some cases to total loss of hold-down capacity.

The test was continued beyond the targeted largest amplitude of the loading history to evaluate full pipe capacity. Testing was terminated when the capabilities of the test set-up were reached.

Figure 12. ±150 mm displacement without pipe clamps.

Figure 13. Test set-up with pipe clamps.

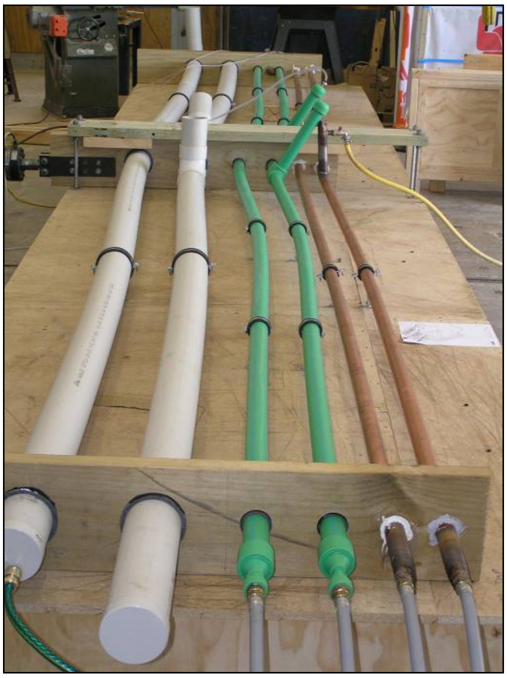

Figure 14. Ultimate limit state inter-storey deflection of ±85 mm with introduced pipe clamps.

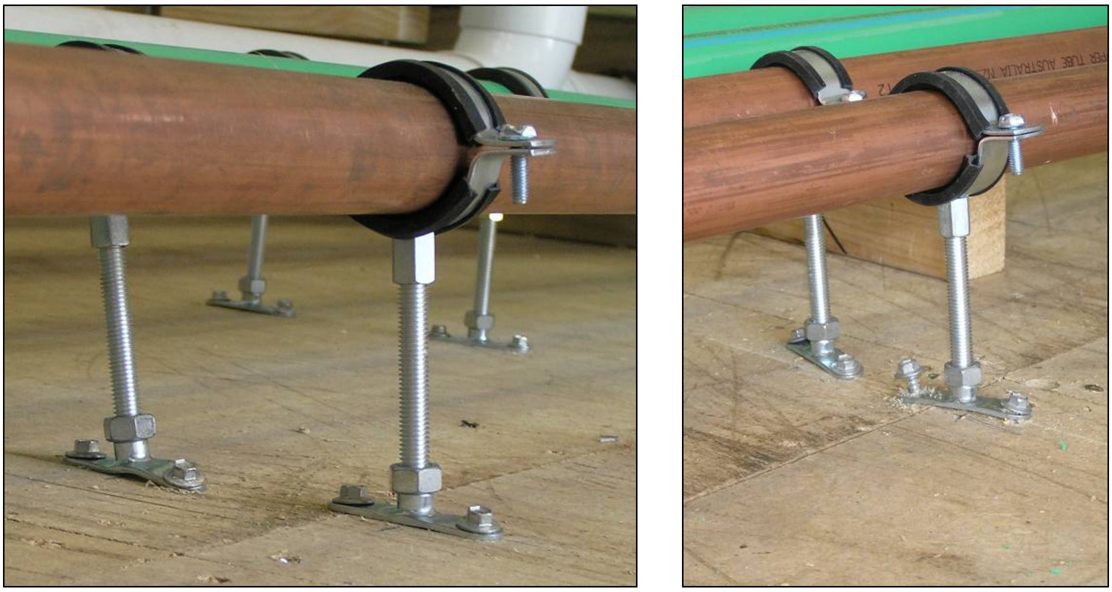

Figure 15. Onset of the pipe clamp loosening.

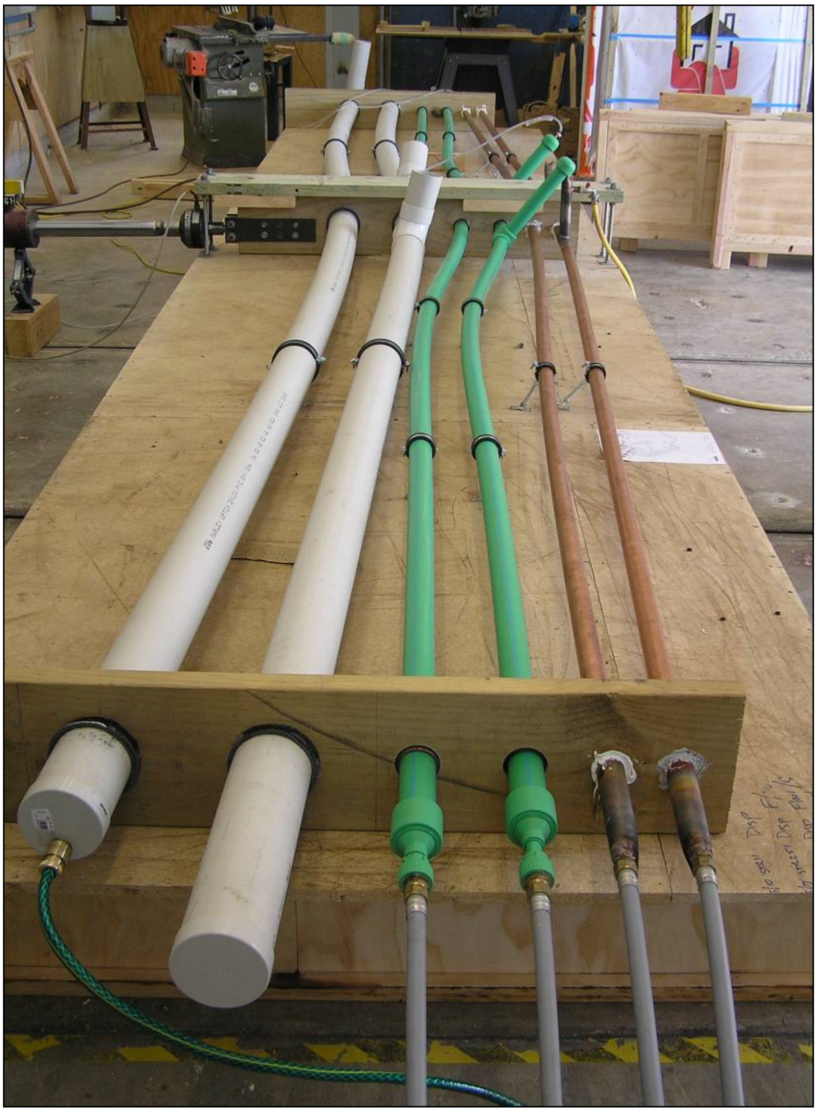

Figure 16. Bending of pipes and the pipe clamps at ±150 mm.

Figure 17. Pipe clamps losing part of the hold-down fixings.

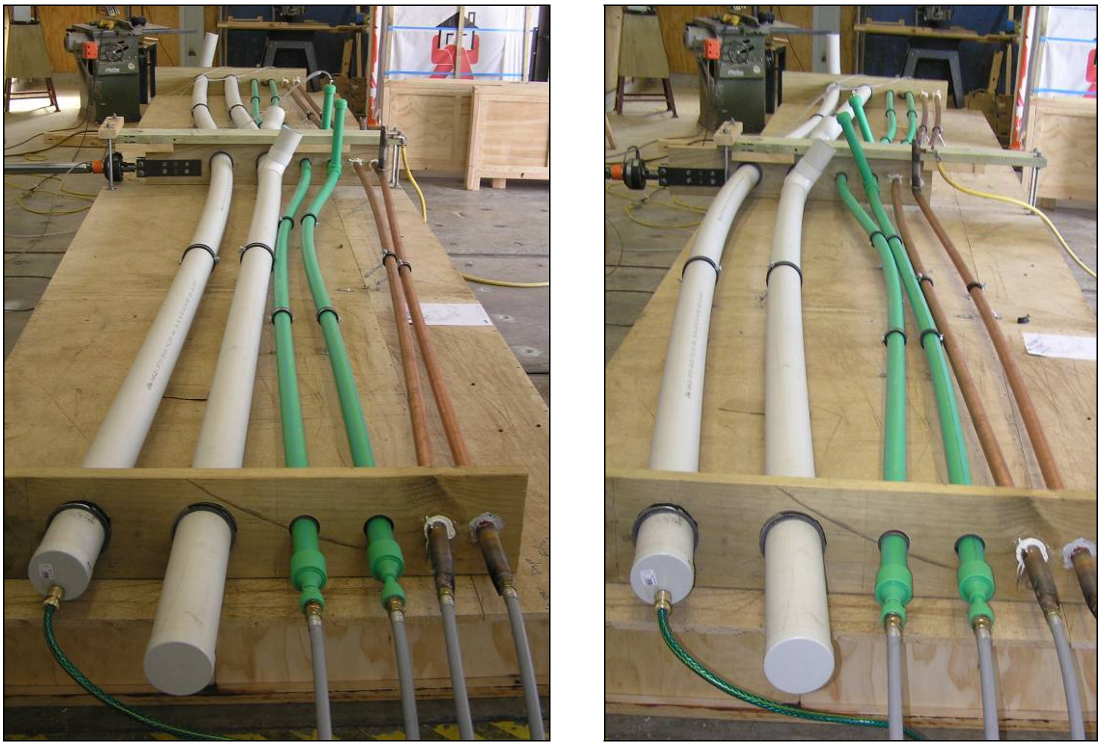

Figure 18. Significant deformation at ±270 mm in side elevation.

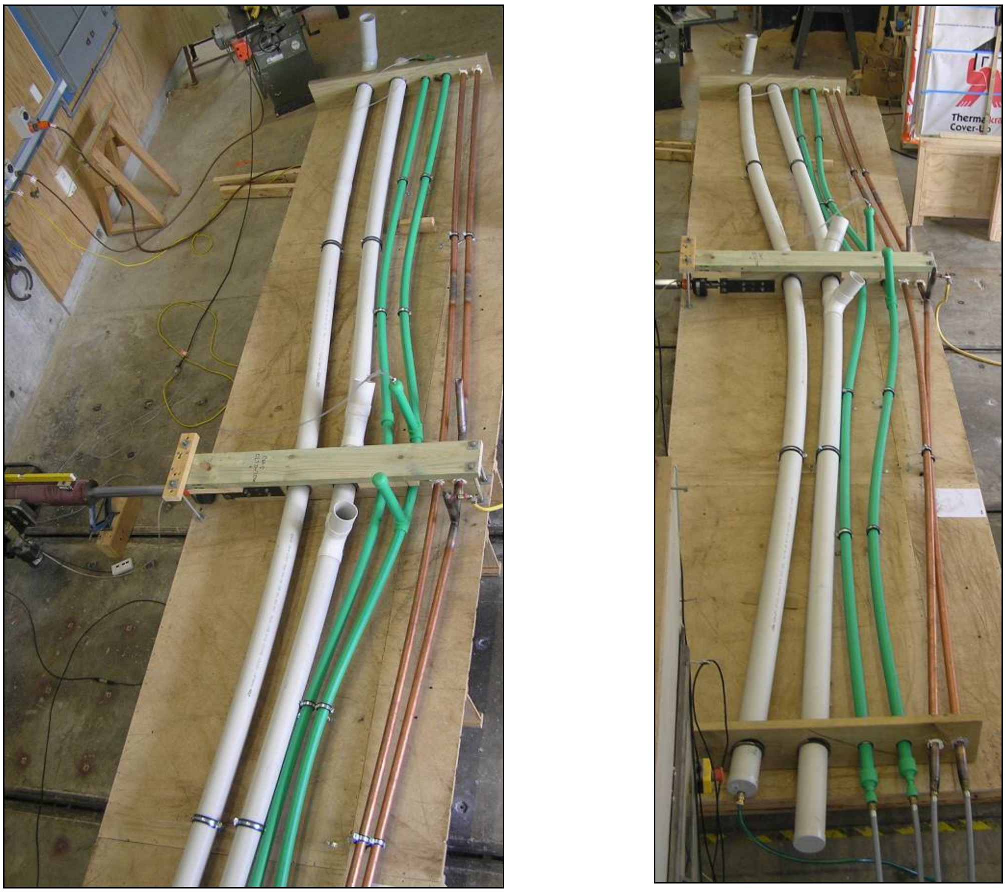

Figure 19. Significant deformation at ±270 mm in plan view.

Figure 20. Kinking of a copper pipe at 'T' joint.

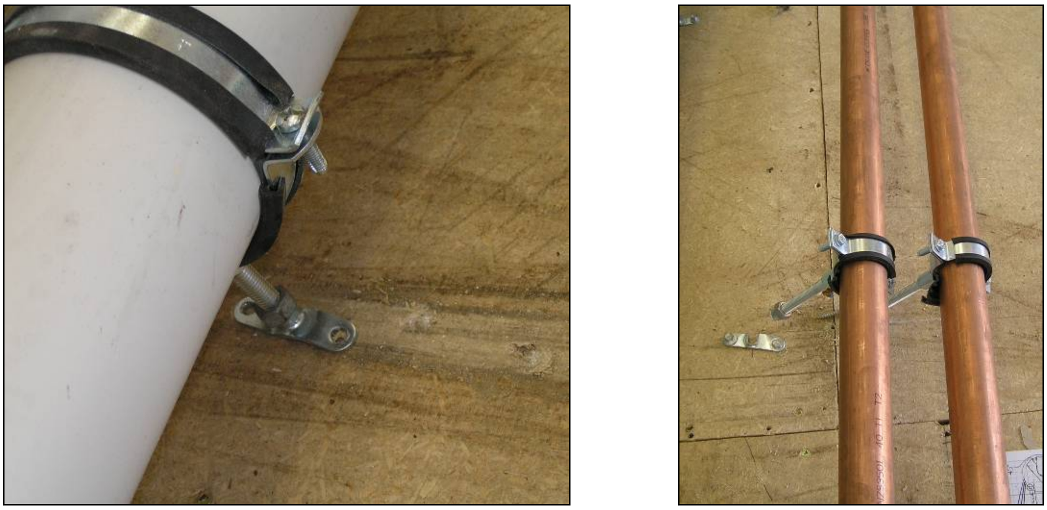

Figure 21. Total loss of pipe clamp hold-down capacity.

9.3 Conclusion

Since the targeted largest amplitude of ±85 mm displacement has not caused failure, the tested pipes are deemed to comply with NZS 1170.5. Moreover, the methodology to verify the dynamic properties for displacement controlled components extending over more than one storey (outlined in Section 6) was found suitable.