responses at different frequencies, and their effective sum is found by applying the SRSS method, also. Usually, each direction of excitation is resolved independently, and the resulting stresses are subsequently added by SRSS.

4.1.6 Excitation Spectra and Frequency Response Spectra

When a structure is to be designed to withstand the vibrations caused by seismic excitation, it will be exposed to an entire spectrum of excitations, of varying amplitudes over the range of frequencies. These excitations may be applied simultaneously, but in fact will act at continually varying amplitudes, so only a few peak inputs usually occur at once. Earthquakes are usually represented for design purposes by *spectral* graphs, which show the *envelope* of the highest values reached at each frequency during the earthquake event. **Figure 4-9** shows such envelope curves, derived from actual recordings at various locations for two earthquakes. **Figure 4-9 (b)** shows recordings of actual above-ground measurements, after being processed through a spectrum analyser. **Figure 4-9 (a)** shows smoothed envelope curves which can be used for design purposes. Typically, earthquake excitation frequencies of concern will fall between 2 or 3 Hz up to 25 or 30 Hz, with the peak ranges generally between 4 and 10 Hz, depending on the sub-soil conditions at the particular site. The input travels through the ground, and peak values may be at about 0.1 g in areas of moderate seismic activity, but are more often in the range of 0.2 to 0.3 g for most CANDU sites, and can reach 0.4 g for sites in some severe areas in the world. This input becomes amplified in the reactor building base slab, sub-structure and internal structure, to input to the reactor structure at its supports at the vault mid-height. If the reactor support and structure has resonances, they will exhibit responses of up to 2 or 3 g assuming moderate damping, and any resonant items mounted on *it* may exhibit responses of 10 g or more.

In order to check seismic adequacy of a complex structure being designed, one would first determine all its resonant frequencies. Then, for each location of interest on the structure, and for each resonant frequency, one would consider the corresponding input excitation from the site spectrum, and calculate the amplitude of its response there. To do this, one would have to prepare mathematical models of the complex structure, determining the amount of amplification exhibited between the ground and each point under consideration. Furthermore, if one were designing the mountings for a component to be installed on that structure, one would have to adapt the model to include that component.

To simplify this process, one can prepare a set of models in advance, covering each level of intervening structure, considering the characteristics of mass, stiffens and damping for each section of structure. One could then plot a graph of *spectral* amplification for each level. The net amplification applied to any given item at any given frequency is then the *product* of the amplifications *at that frequency* for all the intervening levels. The net response of that item would be the ground input at that frequency multiplied by the net amplification. **Figure 4-10** illustrates this concept for a simplified representation of a structure.

In fact, this approach was developed to become the dominant present practice, to prepare *Floor Response Spectra (FRS)* for various levels in a structure. The FRS is *not* a graph of response for a *particular item*, but rather it is an *envelope graph* of the resulting response accelerations for *an entire family* of resonant items mounted at that point on the structure. ie, it is a design tool which *directly indicates* the response which will occur, if one were to mount a sub-system there which has a resonant frequency in that range. Since the sub-assemblies to be mounted might incorporate differing damping factors, a set of FRS curves is normally provided, covering a range of damping factors. **Figure 4-11** shows a typical FRS curve for the CANDU 6 reactor structure.

4.1.7 Analysis of a Structure

As noted above, modern computer programs, such as Patran, can be used to prepare the FEM for a structure, by inputting the geometry and physical properties of the structure and the materials of which it is made. The output is a tabulation of nodes and elements for a Multiple-degree-of-freedom model, and it lists the properties of stiffness, mass and boundary conditions applicable for each element. The analysis program, such as Ansys or Stardyne is then applied to set up and solve the equations of motion for each node. These equations are of the form

$$MA + CV + KX = M_x . a_x ,$$

where M, C and K are three-dimensional mass, damping and stiffness matrices, a_x is the input ground acceleration in the x direction, and M_x is the mass vector in the x direction. This equation defines the motions in the x direction. Similar equations are written and solve in the y and z directions. The outputs of their solution are listings of dominant frequencies and corresponding mode shapes, from which stresses are derived. Stresses in three directions are then combined to provide values of principal stresses and stress intensities, throughout the structure. Graphical representations can also be produced to permit visual understanding of deflected shapes and

Setting up of these equations involves many judgements and assessments to provide appropriate coefficients for the model and the form of excitation to be applied. It is also desirable to eliminate factors which are known to have small influence, so as to minimize the size of the array to be solved, and thereby reduce the amount of the analysis work. Symmetrical structures may be modelled as half-structures, assuming symmetrical motions. Fluids surrounding submerged items must be modelled as attached masses, since a portion of fluid must be displaced by the body when it moves, and a similar portion must be drawn in behind it; this is usually modelled as a body of attached fluid having half the volume of the body itself. A sub-assembly which is known to have a high natural frequency will be modelled as a single element, since it will not respond to the applied excitation and will not exhibit any internal deflections.

In applying the FRS method of analysis, it is recalled that only responses which are close together in frequency will add strongly. Also, only strong responses will have significant consequence, and this is accounted for by multiplying them by a Participation Factor. These factors are accommodated by adding the factored responses vectorially, by applying a method such as the Square Root of the Sums of the Squares (SRSS). Similarly, net stresses acting in the three directions will likely be generated by responses at different frequencies, and the effective sum is found by applying the SRSS method.

4.2 Designing for Improved Seismic Capability - a Design Study

4.2.1 Design Rationale and Behaviour for the Present CANDU 6 Configuration

4.2.1.1 Description of Seismic Characteristics

Figures 4-12 & 13 show the principal features of the *present CANDU 6* Reactor Structure Assembly configuration (RSA). The present design has the RSA supported axially by an *annular support plate* at each end, which flexibly joins each end shield to its *vault wall embedment*; but at one end a ring of *bolts* clamps the plate directly to the embedment, to disable the flexibility there.

The calandria shell (CS) also includes very flexible annular plates, so provides no contribution to axial support; the end shields are joined together only by *the calandria tubes (CT)*. The intention of this concept is to ensure the calandria tubes are not constrained against thermal expansion by either the calandria shell or the vault walls.

A Pressure Tube (PT) passes freely through each calandria tube, but is secured to one end shield by a Positioner Assembly (PA) clamped on its End Fitting (EF). The PA may be secured at either the free or bolted end shield. The highest seismic loads on the PA occur when a Fuelling Machine (FM) is attached to each end of one PT, with the PA at the unbolted ES; only that PT is shown in the Figure. The present PA strength limits the reactor's seismic capability.

At the end without the PA, the support for the other FM is carried through the Rolled Joint (RJ) on the PT. The RJ has higher strength than the PA, but the load here is higher, and may also be a limit on reactor seismic capability.

4.2.1.2 Present Seismic Behaviour

Figure 4-14 shows a simplified schematic representation of the present design. The RSA system may be seen as the two End Shield masses, each with half of all the included masses* attached; the two halves are connected and supported through several paths, but principly the calandria tubes and the bolted support plate. The entire system is supported in the embedment rings in the vault walls. There are two principal modes of seismic vibratory motion: the first mode is due to the mass of the *unbolted* half being driven by the spring of the calandria tubes; the second is due to the total mass of *both* halves being driven as a whole by the spring of the *support plate*.

Their seismic response motions occur at their resonant frequencies, found as:

$$f_1 = (1/2\pi)\sqrt{K_{ct}/(M/2 + M_f)} = 11.7 \text{ Hz},$$

and $$f_2 = (1/2\pi)\sqrt{K_{bs}/(M + M_f)} = 10.6\text{Hz}$$

* The RSA comprises the calandria, calandria tubes, and two end shields, but the mass being supported also includes the enclosed calandria tubes, fuel channels with end fittings, fuel, coolant, moderator, and some shield water attached outside it.

The calculation of the seismic behaviour is shown in **Appendix IV**.

Superimposed on those motions, the *End Shields* themselves have flexibility, whereby they bend in a *dishing* mode (ie, like belleville washers). This third mode is represented in **Figure 4-15**, where the *inner region* of *each* mass moves relative to the outer region. This mode has frequency:

$$f_3 = (1/2\pi)\sqrt{2K_{es} / (2M/4 + M_f)} = 6.6 \text{ Hz}$$

Accordingly, it can be seen that the highest seismic load applied to the *positioner assembly* is due to the *net* acceleration at that end shield, A_{ep}, which is applied to the fuelling machine (plus half the full fuel channel) at *that end*. The net acceleration there is the "dynamic sum" of accelerations due to the first three modes. Since these modes do not occur at the same frequency, their peaks of sinusoidal motion will not occur simultaneously, and this "dynamic sum" is the *Square Root of the Sum of the Squares* (SRSS):

$$A_{ep} = 1.2\sqrt{A_1^2 + A_2^2 + A_3^2}\,.$$

(Note: the 1.2 factor was arbitrarily added in this study, to account for several other, lesser modes acting)

The values for these accelerations are taken from the Floor Response Spectrum (FRS) for the mid-height of the reactor vault, where the reactor is mounted, **Figure 4-11**, for 4% damping, to correspond to the frequencies:

$A_1 = 0.9$ g; $A_2 = 1.0$ g; $A_3 = 1.3$ g;

and $A_{cp} = 2.25$ g,

and the force on the PA is $F_{pa} = 2.25 \times 25\,860 = 58{,}100$ lb.

On the other hand, the highest load on the *pressure tube rolled joint* (RJ) is due to the net acceleration acting on the end fitting at the *non*-positioner end, A_{cn}. This is the SRSS of the net acceleration at the positioner end (A_{cp}) plus that due to the fourth mode, (ie the FM/ PT mode), shown in **Figure 4-16.**

Here, the mass of the fuelling machine at the *non*-positioner end is driven by the spring of the pressure tube itself. Its frequency is:

$f_4 = (1/2\pi)\sqrt{K_{et} / M_f} = 7.4$ Hz, and $A_4 = 1.3$ g.

and $A_{cn} = 1.2\sqrt{A_1^2 + A_2^2 + A_3^2 + A_4^2} = 2.74$ g

and $F_{rj} = 2.74 \times 25\,860 = 70\,800$ lb.

4.2.2 Option 1 for Improved CANDU 6 Configuration - Symmetrical Support

4.2.2.1 Rationale for Change

The proposed accepted approach to design improvement has support plate bolts at *both* ends, to obtain *symmetrical support* and thereby increase *overall* support stiffness. This is shown in **Figures 4-17 and -18.** (Note that this is similar to the Bruce/Darlington design, where symmetrical support is supplied by building both ends rigidly into the shield tank end walls, which are webbed, double-walled steel structures. In the case of Bruce & Darlington, the site has only small seismic excitation, and the shield tank walls can be designed with relatively low stiffness for seismic support, without also applying significant thermal restraint to the calandria tubes.)

In this design option for a CANDU 6, both main, half-reactor masses move at the same frequency, coupled by the calandria tubes. ie, the reactor behaves as one mass.

This whole-body response replaces the first two present modes, and its frequency is:

$$f_5 = (1/2\pi)\sqrt{2K_{bs} / (2M/2 + M_f)} = 14.9 \text{ Hz},$$

where its acceleration is: $A_{ep} = 0.9$ Hz.

The ES mode still acts at 1.3 g, therefore the net acceleration at the positioner end becomes:

$$A_{ep} = 1.2\sqrt{A_5^2 + A_3^2} = 1.90 \text{ g}$$

and the PA force becomes: $F_{pa} = 1.90 \times 25\,860 = 49{,}100$ lb

Similarly, the PT/ FM modes still acts, and the net acceleration at the non-positioner end is:

$$A_{en} = 1.2\sqrt{A_5^2 + A_3^2 + A_4^2} = 2.46 \text{ g}$$

and the RJ force is $F_{rj} = A_{en} \times 25\,860 = 63{,}500$ lb

4.2.2.2 Limitation of Adopting Symmetrical Support for CANDU 6

The values of the two forces of concern, F_{pa} and $F_{rj,}$ are reduced, compared to the present design. However, having stiff support at both ends causes compressive stresses in the calandria tubes, due to the support stiffness inhibiting their thermal expansion. The pressure inside the calandria acting on the end shields causes partially offsetting tensile stresses, but the amount of support stiffening allowed is accordingly limited, to avoid calandria tube column buckling.

The calculation of the net calandria tube stresses σ_{ct} is shown in **Appendix IV**.

The calandria tubes would expand 0.0326 in (each end) if unrestrained, but the vault in which the support plates are mounted only expands 0.0120 in. The axial stiffness of the calandria tubes (half length) is 32.28(10^6) psi, while that of each support plate has 25(10^6) psi. The *resulting nominal average compressive* stress in the calandria tubes is about 2,110 psi. The internal calandria pressure applies about 1,950 psi tension *average*, (it is largest at the bottom, but zero at the top). When the additional restraint stress due to the end shield's thermal deformation is added*, the tubes near the top edges have a *net compressive stress of about 2,660 psi or more* acting on them. This is undesirable for *fuel channel creep sag.*

(* The end shield dishes thermally, due to its calandria-side tube sheet being cooler than the fuelling side tube sheet. Central calandria tubes are therefore caused to stretch while peripheral tubes are compressed, with tube loads varying radially between the two extremes, and these loads are self-balancing across the total area of the end shield. These added loads due to the end shield-thermal deformation can accordingly be superimposed on the loads calculated above. Loads from previous CANDU 6s are used, since the temperatures are the same: edge CTs have about 2,500 psi compressive stress added and central CTs have about 1,500 psi tensile stress added.)

4.2.3 Option 2 - Straight Calandria Shell Configuration

4.2.3.1 Rationale for Change

The symmetrical design change in Option 1 requires a difficult balance between obtaining high stiffness in the annular support plates for reduced seismic response, and limiting the stiffness to avoid thermal restraint. The straight calandria shell design, shown schematically in **Figures 4-19 & 4-20**, carries virtually all the axial seismic load to overcome this limitation. Furthermore, because the shell has higher thermal expansion than the calandria tubes, it tends to push the vault walls apart and tends to apply *tension* to the calandria tubes. The support plate stiffness can accordingly be selected to raise the frequency of the fifth mode sufficiently to reduce PA and RJ loads without applying compression to calandria tubes. The calandria shell stiffness is higher than that of the vault walls, and its expansion will dominate this condition.

In this design, the joint at the annular plate is made much stiffer than in the bolted version, and is in fact totally eliminated, by bridging the main shell over to join directly to an extended end shield outer diameter. The frequency found for f_5 is raised to be:

$$f'_5 = (1/2\pi)\sqrt{2K'_{bs} / (2M/2 + M_f)} = 23.1 \text{ Hz}$$

where its acceleration is now: $A_5 = 0.8$ g, so net acceleration is reduced only a little:

$$A_{ep} = 1.2\sqrt{A'^2_5 + A_3^2} = 1.83 \text{ g}$$

and the PA force is reduced a little:

$$F_{pa} = 1.83 \times 25\,860 = 47{,}400 \text{ lb}$$

At the non-positioner end,

$$A_{en} = 1.2\sqrt{A'^2_5 + A_3^2 + A_4^2} = 2.40 \text{ g}$$

and the RJ force is $F_{rj} = 2.40 \times 25\,860 = 62{,}300$ ib

(As support plate stiffness approaches the stiffness of the vault wall itself (about $100(10^6)$ lb/in, account should also be taken of the response motion of the walls themselves, whereby they also bend in a dishing mode. In that case, the inner portion of their mass, surrounding the embedment, will also be in motion and it must also be included in this seismic representation. This might lower the frequency a little, but the net response acceleration level will be no different, because the FRS is very flat there.)

The three designs are compared in the Table in **Figure 4-21.**

The most significant benefit of the straight-shell design in Option 2 is that the calandria tubes are placed in modest *tension*, rather than compression, due to thermal loads. The net stress due to combined axial thermal loads is about 1,000 psi *tension around the edges*; this will be increased to about 2,000 psi tension in the centre, due to end shield thermal "dishing". On the other hand, the calandria ***shell*** now carries a modest 6,000 psi compression stress, in forcing the supports apart. The symmetrical-bolted version of Option 1 has about 190 psi compressive stress near the edges and about 1,080 tension near the centre.

Furthermore, because the calandria ***shell*** carries nearly all of the loads, the calandria ***tubes*** have virtually no other stresses added to them due to other conditions such as thermal transients, seismic events or postulated extreme pressure cases. In comparison, in both the current design and the symmetrical change proposed in Option 1, seismic loads will add some extra stress, but postulated extreme pressure accidents (eg 100 psi) will add about 15,000 psi tension to the calandria tubes. Furthermore, the weld joining the present *calandria* annular plate to the main shell is presently the limiting weak area for the latter case (due to constraint of shell deflection); but this joint and its limitation are totally *eliminated* in the straight shell design.

The proposed construction for this concept is shown in the detail in **Figure 4-22**. Compared to the present design, **Figure 2-3**, this figure also reveals that the straight profile permits elimination of the rings of bolts holding the end shield to the vault embedment. It also allows elimination of the steel shielding slabs and water cooling pipes embedded in the vault concrete, used to protect the concrete from nuclear heating due to radiation emanating from the annular plate.

4.2.3.2 Drawback for the Straight Shell Option

After defining the new concept, the means of constructing it was considered. Compared to the present design, it appeared it would only be necessary to increase the outside diameter of the end shield to match the main shell diameter, then extend the main shell plate to join to the end shield. However, the very real problems of assembly sequence, alignment and access for welding raised formidable practical concerns. In order to apply code-acceptable, full-penetration welds, the weld joining the support shell to the extended end shield outer tube sheet (the weld identified as #7 on the section in **Figure 4-22**) requires a close fit-up so it can be made on-sided; that means the weld of the main shell extension to this tube sheet must be made first, and the tube sheet face machined flat. Therefore, all the internal members comprising the sub-shell (retained to maintain the original reflector fluid profile) and the webs (added to stiffen the edge of the extended end shield) must be welded in first. (See weld sequence numbers #1- #5.) The welder is able to work inside the end shield for those members. Furthermore, in order to make weld #6 then, the shell must be slid over the main shell extension, therefore there must be a subsequent seam weld in the main shell, weld #11. Furthermore, the calandria shell extension requires access openings, to permit making of welds on those internal welds, and it becomes a series of bars ("dog-bones"), requiring more cutting operations. All these extra welds add significant cost, since they run around a 24 metre perimeter. However, the most significant difficulty was that these welds added about 10 weeks to the manufacturing schedule, for a key item on the plant construction schedule.

This excise demonstrates the difficulty in changing established designs; designs with technical superiority aren't necessarily chosen unless they meet *all* the requirements for the design. In the present atmosphere in the nuclear business, the length of the construction schedule is of extreme importance.

4.2.4 Comparison of Designs

Summary of straight calandria shell option, compared to "Bolt Both Ends" option

- Seismically slightly better than "Bolt Both Ends" design: - 20% vs 15% gain over current design (estimated values were confirmed by FEM analyses)

- Adds tensile bias to calandria tube thermal stress - reduces creep sag

- Removes calandria shell annular plate & shell - Current limiting stress region for in-core LOCA accident - ("Dummy" notch included to retain reflector profile)

- New "Torsion Box Ring" around end shield increases end shield stiffness - Reduces seismic response displacement in end shield (*marginal gain*)

- Deletes Embedded cooling pipes in concrete around end shield embedment, also steel shielding slabs inboard of end shields & lead wool shielding in annular support shell

- Minor added one-time cost for design, FEM & A, mfg liaison, (reactor, civil, process), compared to "Bolt Both Ends"

- Increases RSA cost by 10%, delays schedule by 10 weeks

4.2.5 Complex Systems - Experimental Verification - Seismic Testing

Testing is rarely done for rigidly-jointed structures or assemblies, since their behaviour can usually be predicted accurately by modern modelling methods. Furthermore, even for mechanical sub-assemblies having resonant internal components, the behaviour of the internal parts may not be of concern, because it will have negligible effect on the behaviour of the more massive system as a whole. However, testing is necessary when the internal components' mechanical or electrical functioning is important, rather than simply structural survival, such as for flux detector and shutoff units. Their special testing procedures are discussed in **Sections 6-6**, below.

There are also circumstances where analysis cannot accurately cope with the jointing details, where testing is needed. An example of this is the case where there are loose gaps in the connections between main sub-systems which allow relative motion, especially if there is some form of damping between them. In some cases, the true behaviour can be adequately understood by making simplifying assumptions for analysis and then subsequently assessing how the result would be affected if the simplification were removed. For example, if a gap is larger than the amplitude of the input motion, there is effectively no connection to the "downstream" part, and the "upstream" part can be analysed on its own.

Such would be the case for a simply supported beam with a large gap at a mid-span support, as shown in **Figure 4-23**. If it were then excited by an input which had a peak at its natural frequency, its response amplitude would grow until it made contact at the gap, when it would start to transmit the vibration to that support, and it would then become a different system. The beam would now have three-point support, and the "downstream" part of the structure would be attached. Since both the beam and the overall system would have different natural frequencies, the beam would no longer be responsive to the original peak input, and the response amplitude would die away again; the system would revert to being only the "upstream" part, where it would again start to respond, to repeat this process. Therefore, the behaviour of this gapped system would be characterized by a cyclic increase and decrease of the amplitudes of response at alternately different frequencies, such that it would not reach maximum amplitude. In effect, this system is partly damped.

Similarly, if a gap has heavy viscous damping, or friction damping, it will behave as if it is partly coupled, and the system behaviour is very difficult to predict.

Testing may become necessary in such cases. The fuelling machine and its variable-geometry carriage, bridge and column support system is an example of this. That system is described in **Section 7**, below.