SEISMIC DESIGN OF LNG STORAGE FACILITIES - CURRENT CANADIAN PRACTICE

by

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ABSTRACT

Seismic design for LNG storage facilities under the current Canadian Standard Z276-M1981 is reviewed. The following topics are discussed: containment systems; seismicity assessment methods for determination of design response spectra; dynamic analysis models; and the need for design consistency.

1. **INTRODUCTION.** The authors have been involved in seismicity, structural design and safety assessments for a number of proposed and existing Liquefied Natural Gas (LNG) storage facilities in western Canada. This paper reviews the seismic design provisions of the current Canadian Standard(1).

2. **REVIEW OF DESIGN CONCEPTS.** Liquefied Natural Gas (LNG), the major component of which is methane (CH_4), is stored at about -163°C and a vapour pressure of about 4.0 kPa above atmospheric pressure in large, double walled, insulated tanks up to 153 600 m^3 capacity(19). The liquid has a specific gravity of about 0.48 and very low viscosity, about 0.14 that of water. LNG occupies only about 1/630 of its volume as a gas at normal temperatures; hence its usefulness when storage and transport are involved.(2) The inner tank temperature of -163°C makes normal carbon steels unsuitable. The most widely used materials are 9% nickel steel and aluminum alloys, although prestressed concrete, with and without membrane lining, is also used.

Under the Standard, Z276, a secondary impoundment system is required to "minimize the possibility of accidental discharge of LNG...endangering adjoining property or important process equipment and structures, or reaching waterways...". Some typical LNG tank configurations which would satisfy the Standard are given in Figure 1.

3. **PRESENT SEISMIC DESIGN REQUIREMENTS OF THE CANADIAN STANDARD.** This section briefly reviews the Standard provisions for seismic design, most of which are contained in Section 5.1.3, entitled "Seismic Design". The opening Clause (5.1.3.1) requires that seismic loads be considered in design and that a site investigation, including geological and seismic characteristics at the LNG facility and the surrounding region, be performed to determine "seismic potential" and response spectra. Clause (5.1.3.2) outlines the requirements in more detail.

The following Clause (5.1.3.3) introduces the two tier seismic design basis: "Safe Shutdown Earthquake" (SSE), and "Operating Base (Basis?) Earthquake" (OBE), defined probabilistically in part (a) of the clause as ground motions with mean recurrence intervals of 10,000 and 475 years respectively. Part (b) of the clause introduces an alternative definition of SSE and OBE, as follows: "...where the uncertainties are difficult to quantify because of the lack of geological data, the deterministic approach shall be used where the SSE is the event which produces the maximum credible ground motion at the site based upon the seismology, geology, seismic and geologic history of the site and region, and where the ground motions for the OBE shall be one half of those determined for the SSE". Procedures for determining OBE and SSE are discussed in Section 5 of this paper.

Clause (5.1.3.4) is a classification clause designating the structures and systems subject to the two tier seismic loads: the LNG container and its impoundment system; those system components required to isolate the LNG container and maintain it in a safe shut-down condition; and the fire protection system. This is not the complete list; piping is referred to later (7.1.2). The classification indicates that the primary container must be designed for simultaneous full LNG level hydrostatic and earthquake induced hydrodynamic forces. The secondary impoundment (cryogenic outer tank or separate dike) is also designed for concurrent hydrostatic and hydrodynamic forces.

Clause (5.1.3.5) requires an LNG container to be "designed for the OBE and a stress limit check made for the SSE". By inference, the term "LNG Container" includes the other items classified in Clause (5.1.3.4). Stresses for OBE are given in standards(3,4) which follow established design procedures and in general confine maximum allowable stresses to within the elastic range. Stress limits for the SSE are the yield stress in tension and the critical buckling stress in compression. Depending on the relative intensities of ground motion and the allowable stresses, either the OBE or the SSE could actually govern structural section sizing.

Clause (5.1.3.6) gives some broad guidance for the dynamic analysis of the LNG container. This will be discussed in more detail in Section 6.

4. **SEISMIC FAILURE MODES AND EFFECTS.** The inner tank designed according to the Standard as discussed in Section 3 will survive an SSE level event. The secondary dike is designed to provide a redundant or backup containment system. The protection for the public, in the unlikely event of an inner tank failure, lies in the dike limiting the size of the vapour plume from LNG exposed to the atmosphere.

The dike is designed for concurrent hydrostatic and earthquake-induced hydrodynamic forces, and thus should retain its integrity since it will contain no liquid during most of the event, and therefore be subject to much lower forces. If the dike is made of a different material from the inner tank (say pre-stressed concrete or earth vis a vis 9% nickel steel) there is some small added protection in that structural response frequencies and material behaviours will differ; the combination of circumstances which fail a steel tank will not necessarily fail an earth embankment.

At the OBE level, the overall safety objectives of the Standard are to maintain operating capability, safety of the operators and the protection of the immediate plant area.

5. **THE SEISMICITY ASSESSMENT.** The Standard requires a site specific geological and seismic investigation including the examination of surface faulting at and around the site, the potential for liquefaction and the wave transmission characteristics. The aim of the investigations is to produce site specific Design Response Spectra (DRS) corresponding to OBE and SSE levels. The Standard appears to allow probabilistic or deterministic methods in determining the SSE.

For many Canadian sites, we tend to favour a probabilistic approach such as that known generally as the Cornell Method(5). The method combines the regional historical distribution of earthquakes with the geologic and tectonic causes (when known), by grouping earthquakes in area or line zones. The frequency of occurrence of earthquakes in the zones is used together with attenuation relations, which describe the decay of ground motion with distance, to evaluate ground motion parameters such as horizontal peak ground acceleration (PGA) or peak ground velocity (PGV) at the site as a function of probability of exceedence.

The advantage of a probabilistic as opposed to deterministic approach lies in providing input for system risk analyses and also a framework for evaluating the effects of uncertainties. Atkinson and Charlwood(6) discuss the use of the Cornell method at low probabilities, and conclude that (i) the results are sufficiently stable for engineering purposes; and (ii) any available deterministic data are best considered in the probabilistic analysis in which the treatment of uncertainty associated with specific scenarios is handled formally. Therefore, there appears to be no need to use the deterministic option referenced in the Standard.

For preparation of DRS for firm ground conditions, the conventional approach is to scale standard or average DRS (eg. as given in Reference 7 or 8) to the appropriate PGA value determined from a seismicity analysis. These standard DRS are derived by compounding many different earthquake spectra from a range of sites, source distances, magnitudes, etc. In our opinion, it is preferable to use procedures that preserve more site specific information, such as frequency content. For example, McGuire(9) describes an approach which uses frequency dependent attenuation relations for DRS ordinates directly.

The Standard requires examination of the "potential for soil liquefaction". It is desirable to treat the liquefaction potential in a probabilistic context to maintain overall design consistency. This aim requires a systematic combination of soil properties with both intensity (PGA) and expected duration of shaking (or magnitude). If the liquefaction potential is addressed in this fashion, it can then be stated as an annual probability, to be compared directly with the design annual probability of structural failure. The probability of liquefaction can be reduced by ground treatment if necessary.

Finally a designer may need time histories, particularly in the design check for the SSE, since non-linear analysis may be involved. The most likely source zones and magnitudes of significant events can be identified from a Cornell analysis, and a suite of typical earthquake accelerograms can then be selected from available or synthesized records. For consistency with the DRS, these records require scaling such that the spectra of the selected records would equal or exceed those of the DRS at least in the frequency range of interest.

6. **STRUCTURAL ANALYSIS FOR SEISMIC LOADS.** The theoretical response of a liquid filled, cylindrical tank to seismic excitation is complex. Housner(10,11) developed the original simplified mechanical analogue in which fluid forces were separated into a "convective" component, related to the sloshing of the liquid in the tank and an "impulsive" component, associated with the dynamic response of the tank plus an attached, equivalent mass of fluid (Figure 2(a)). The original analysis assumed the containment vessel was rigid. A re-examination of the analytical methods by Veletsos and Yang(12) showed that when the flexibility of the tank was taken into account the dynamic loads from the seismic response were substantially greater than for a similar rigid tank. Housner and Haroun(13) subsequently produced the equivalent mechanical analogue for the flexible tank, shown in Figure 2(b).

For typical LNG tanks, the natural period of the "convective" spring-mass system (sloshing) is about 5-6 seconds, while that of the impulsive system (tank fluid response) is about 0.5 seconds. The simplified spring-mass model can be used with DRS and a modal analysis method to obtain the bending moment and shears at the base of the tank section.

The size of the calculated dynamic structural loads depends on the choice of damping factor. A decrease in allowable damping is equivalent to an increase in the DRS level or a decrease in allowable stresses. These three factors together dictate the structural earthquake resistance. The impulsive damping is essentially that of the tank material; i.e. steel or prestressed concrete. The LNG Seismic Review Panel on the Little Cojo Bay project(14) recently recommended that damping of 3% be used for analysis of tanks in conjunction with essentially elastic behaviour at an OBE level. Greater damping values were allowed for larger earthquakes, along with higher permitted stress levels. For convective damping, the California Public Utilities Commission(15) recommends a value of 1/8% for sloshing wave action.

The modal responses are usually combined using the square root of the sum of the squares of the individual modal responses (absolute sum for closely spaced modes) and the spatial components (two horizontal and one vertical) are similarly summed.

The analysis yields the base shear and the bending moment on the tank as a whole, to be resisted by tension and compression stresses in the tank wall. The tensile stress limits come directly from the code and material properties. The compressional stress limits for OBE level excitation come from the specified codes which assume certain stress reduction factors for buckling. For the SSE compressional stress, an estimate of the actual critical buckling stress is made; this includes the stabilizing effects of hoop tension.

A further element of tank design, that of freeboard allowance for "sloshing" of the liquid, comes directly from the calculated response of the convective elements in the mechanical analogue.

The analysis can accommodate flexible foundations. The use of the mechanical analogue combined with the equivalent lumped, foundation spring-mass parameters, given by Whitman(16) provides an adequate solution (Figure 2(c)). The possibility of uplift may be important for relatively lightweight steel tanks. Recent approaches to LNG tank design(4) recommend that uplift be prevented. It is possible to analyze tank uplift under dynamic loads using a non-linear procedure, and then design accordingly. However, due to the complexity of such procedures, it is generally preferable to avoid this approach by ensuring no uplift occurs.

7. **DESIGN METHODOLOGY.** The general aim of a code or standardization is to ensure an acceptable level of safety within sensible economic limits. Most codes approach the aim through a series of internal safety requirements at various detailed points of application of the code. A necessary requisite to achieve the global aim is that the internal provisions be consistent relative to one another.

At the OBE level the Standard provisions for the storage system and piping are reasonably consistent. The OBE level forces, when combined with relatively conservative working stress levels inherent in construction standards (eg. OBE spectra with API 620 or CSA CAN3-A23.3 for the tank and ANSI B31.3(18) for piping), result in structures capable of withstanding base accelerations considerably higher than the OBE design acceleration(17).

At the SSE level, the degree of internal consistency is more difficult to assess, due to the complex nature of ultimate behaviour. Model or prototype testing may be beneficial in checking theoretical predictions.

When provisions to protect the site against liquefaction or slope instability are considered, or when an earthen structure for secondary containment is designed, the issue of consistency between approaches is again complex. Earth embankments are traditionally designed using base "coefficients" and pseudo-static design methods. The correlation between base "coefficients", and actual base accelerations as represented by OBE and SSE levels needs to be addressed.

Another aim of internal consistency should be to avoid compounding safety factors. For example, the ultimate seismic resistance of the tank depends on three factors (DRS, damping, allowable stresses). If the DRS, damping and stress levels are all conservative by a factor of 1.5, then the overall design will be conservative by a factor of about 3.4 (1.5^3). Anderson and Bachman(20) give a more detailed example in which they show that this compounding of safety factors results in a tank designed under a draft California LNG code having a higher effective factor of safety than one designed under the U.S. Nuclear Regulatory Commission criteria for nuclear power plants.

8. **SUMMARY AND CONCLUSIONS.** The design philosophy of the Standard aims

to preserve normal operation under moderate earthquakes and to protect certain critical elements to ensure public safety from much stronger seismic attack. Under very strong seismic attack, provision of a redundant impoundment system will provide additional public safety.

AUTHORS' NOTE

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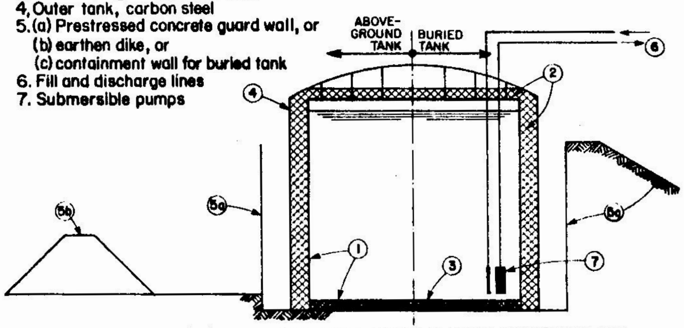
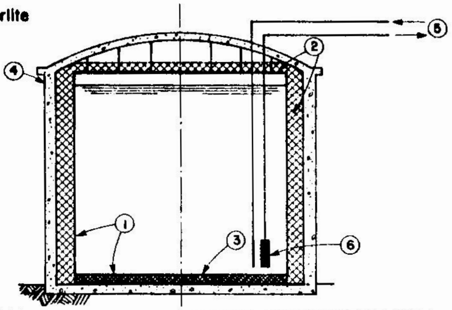
1. Inner tank, 9% nickel steel
2. Wall and roof insulation, perlite
3. Loadbearing insulation to base
4. Outer tank, carbon steel
5. (a) Prestressed concrete guard wall, or
   (b) earthen dike, or
   (c) containment wall for buried tank
6. Fill and discharge lines
7. Submersible pumps



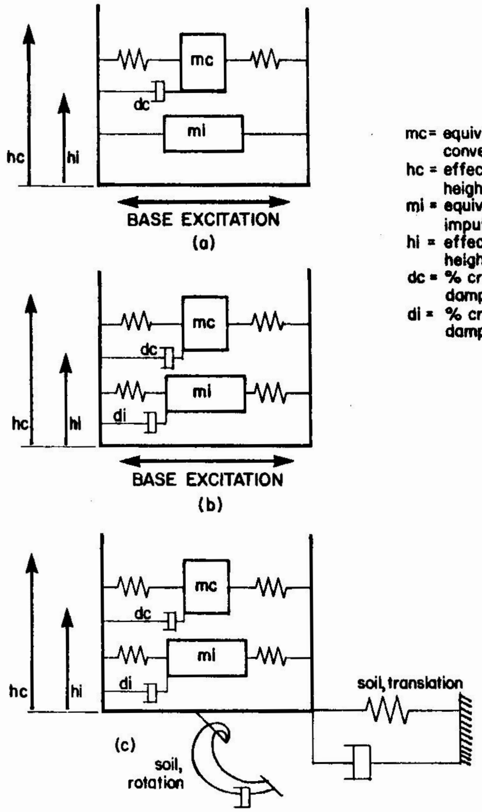
A. THIN OUTER WALL DOUBLE TANK CONSTRUCTION

1. Inner tank, 9% nickel steel
2. Wall and roof insulation, perlite
3. Loadbearing insulation, foamed glass
4. Integral prestressed concrete containment wall
5. Fill and discharge lines
6. Submersible pumps



B. INTEGRAL PRESTRESSED CONCRETE CONTAINMENT WALL

FIG. 1 TYPICAL TANK CONFIGURATIONS



mc = equivalent convective mass
hc = effective height of mc
mi = equivalent impulse mass
hi = effective height of mi
dc = % critical damping, convective
di = % critical damping, impulsive

FIG. 2 DYNAMIC MODEL FOR SOIL/STRUCTURE/FLUID INTERACTION