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Review of Code Provisions on Design Seismic Forces for Liquid Storage Tanks

by

Dr. O. R. Jaiswal
Department of Applied Mechanics
Visvesvaraya National Institute of Technology
Nagpur

Dr. Durgesh C Rai
Dr. Sudhir K Jain
Department of Civil Engineering
Indian Institute of Technology Kanpur
Kanpur

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Abstract

It is well recognized that liquid storage tanks possess low ductility and energy absorbing capacity as compared to the conventional buildings. Accordingly, various design codes provide higher level of design seismic forces for tanks. In this article, provisions of IBC 2000, ACI, AWWA, API, Eurocode 8 and NZSEE guidelines are reviewed, to assess the severity of design seismic forces for tanks vis-à-vis those for buildings. It is seen that, depending on the type of tank, design seismic force for tanks can be 3 to 7 times higher than that for buildings. Based on the comparison of provisions in these documents, various similarities, discrepancies and limitations in their provisions are brought out. At the end a brief description of Indian code is given along with a few suggestions to remove the inadequacies in Indian code.

1. INTRODUCTION

Seismic safety of liquid storage tanks is of considerable importance. Water storage tanks should remain functional in the post earthquake period to ensure potable water supply to earthquake-affected regions and to cater the need for fire fighting. Industrial liquid containing tanks may contain highly toxic and inflammable liquids and these tanks should not loose their contents during the earthquake. Liquid storage tanks are mainly of two types: ground supported tanks and elevated tanks. Elevated tanks are mainly used for water supply schemes and they could be supported on RCC shaft, RCC or steel frame, or masonry pedestal.

Failure of tanks during Chilean earthquake of 1960 and Alaska earthquake of 1964 led to beginning of many investigations on seismic analysis of liquid storage tanks. Following two aspects came to forefront:

(a) Due consideration should be given to sloshing effects of liquid and flexibility of container wall while evaluating the seismic forces on tanks.

(b) It is recognized that tanks are less ductile and have low energy absorbing capacity and redundancy compared to the conventional building systems.

Studies focused on the first aspect resulted in the development of mechanical models of tank by Housner (1963) and Veletsos (1974), which represented tank-fluid system in a more realistic fashion. Many investigations followed along this line to further refine these mechanical models to include effects of flexibility of soil (Hori (1990), Veletsos et. al. (1992)) and base uplifting of unanchored tanks (Malhotra (1997)). Further studies have provided more simplifications to these mechanical models (Malhotra (2000)). Most of the design codes use these mechanical models to represent dynamics of tank-fluid system, which are applicable to ground supported as well as elevated tanks.

The second aspect which is related to low ductility and redundancy in tanks as compared to the conventional buildings, has been dealt with in a rather empirical manner. Lateral seismic coefficient for tanks is generally taken higher than for the buildings. Wozniak and Mitchell (1978) state "*… the high value of lateral seismic coefficient for tanks in comparison with buildings is appropriate because of the low damping inherent for storage tanks, the lack of nonstructural load bearing elements, and lack of ductility of the tank shell in longitudinal compression*". Most of the design codes do follow this approach and assign higher design seismic action for tanks as compared to buildings. How high this design action should be, is perhaps decided on ad-hoc basis or based on past experiences, however, it is influenced by type of tank, supporting subgrade, type of anchorage to tank etc. Basically it depends on how good ductility and energy absorbing capacity a particular type of tank can provide. For elevated tanks, ductility, redundancy and energy absorbing capacity is mainly governed by the supporting structure, which could be in the form of a RCC shaft, RCC frame, Steel frame or even masonry pedestal.

This article presents an assessment of design seismic force for tanks *vis-à-vis* design seismic force for buildings as mentioned in the following documents:

(a) IBC 2000
(b) ACI Standards ACI 371 (1998) and ACI 350.3 (2001)
(c) AWWA D-100 (1996), AWWA D-103 (1997), AWWA D-110 (1995) and AWWA D-115 (1995)
(d) API 650 (1998)
(e) Eurocode 8 (1998)

(f) NZSEE guidelines and NZS 4203:1992

It may be noted here that IBC 2000, ACI, AWWA and API standards are from USA. The quantification of design seismic action in ACI, AWWA and API standards is in a different fashion than IBC 2000. However, FEMA 368 (NEHRP 2000) has provided modifications to these quantifications to bring them in conformity with provisions of IBC 2000. In the present article, provisions of ACI, AWWA and API standards will be discussed along with the modifications of FEMA368. Similarly, in New Zealand, the NZSEE recommendations (Priestly et. al., 1986) on seismic design of tanks, is being presently revised by a study group to bring it in line with New Zealand loading code NZS 4203:1992. The outline of the procedure proposed by this study group is given by Whittaker and Jury (2000). In the present article, procedure described by Whittaker and Jury is considered along with NZS 4203:1992.

The assessment of design seismic force for tanks is presented in terms of design response spectra. This assessment is done with respect to corresponding design seismic force for buildings. Such a comparative assessment helps in knowing how severe design seismic action for tank is, as compared to that for a building under similar seismic exposures. First, provisions on design seismic action for tanks described in the above-mentioned documents are discussed, followed by a comparison of design seismic actions from various codes. At the end a brief description of Indian Standard, IS 1893:1984 is given. Inadequacies of IS 1893:1984 in quantifying suitable seismic design forces for tanks are brought out and a few modifications are proposed to remove these limitations.

2. IBC 2000

International Building Code (IBC) 2000 does provide provisions for certain types of non-building structures which include tanks. For buildings, the seismic base shear is given by $V = C_s W$, where, W is the effective seismic weight. Seismic response coefficient or base shear coefficient, C_s should be minimum of the following two values $C_s = \frac{S_{DS}}{R/I}$ or $C_s = \frac{S_{D1}}{(R/I)T}$, where S_{DS} and S_{D1} are the design spectral response accelerations at short periods and 1 second period, respectively; I is importance factor; R is response modification factor and T is the fundamental time period of building. The minimum value of C_s should not be less than 0.044 S_{DS} I. IBC suggests a value of R = 8.0 for buildings with ductile frames. For most of the buildings, importance factor, I = 1.0. Figure 1 shows the variation of base shear coefficient, C_s = (V/W) with time period. The values of S_{DS} and S_{D1} are taken as 1.0 and 0.6 respectively, which correspond to S_D = 1.5, F_a = 1.0, S_1 = 0.6 and F_v = 1.5 with site class D.

For tanks, due to low ductility and redundancy, low values of R are specified. Table 1 gives details of various types of tanks mentioned in IBC along with their R values. Four values of R are specified, i.e. R = 1.5, 2.0, 2.5 and 3.0. For most of the tanks the value of importance factor I will be I =1.25. However, for tanks containing highly toxic materials, importance factor could be I =1.5. The expression for base shear of tank is same as that for building with suitable values of R and I. For tanks, the minimum value of C_s should not be less than 0.14 S_{DS}I as against 0.044 S_{DS}I for buildings. For ground-supported tanks (i.e., at-grade tanks), IBC suggests to include the effects of sloshing. Similarly, for elevated tanks (i.e., above-grade tanks), IBC states that when sloshing mode period of the stored liquid is within 70 percent to 150 percent of the fundamental period of the supporting structure, the effects of sloshing shall be included in the design of tank and supporting structure. However, IBC 2000 does not provide any particular details on evaluation of sloshing or convective mode forces. Thus, the values of R specified for tanks can be considered only for impulsive modes. The variation of base shear coefficient (BSC) for tanks, with time period is also shown in Figure 1. It is seen from this figure that depending on the type of tank, base shear coefficient is 3 to 7 times higher than that of a ductile building. The ratio of base shear coefficient of tank and building, (BSC_{tank}/BSC_{bldg}), plotted in Figure 2, directly indicates how severe design base shear for tank is with respect to a ductile building (R = 8.0). The effect of response reduction factor of tank is seen up to 2 sec. For time period greater than 3 sec, all types of tank have same base shear coefficient, which is about four times that for a ductile building.

3. ACI STANDARDS

ACI 371 and ACI 350.3 describe provisions for seismic design of liquid storage concrete tanks. ACI 371 deals with pedestal supported elevated RCC tanks only; on the other hand, ACI 350.3 deals with ground supported as well as elevated tanks. Further, ACI 371 describes consideration of impulsive mode only, whereas, ACI 350.3 has provisions for impulsive as well as convective modes. The quantification of design seismic action in these ACI standards is in a manner different from IBC 2000. In order to bring these quantifications in conformity with IBC 2000, FEMA 368 has suggested modifications to the base shear coefficient expressions of ACI standards. Prior to study of various provisions of ACI standards, it will be appropriate to review the modifications suggested by FEMA 368. Table 2 gives the details of base shear coefficient expressions of ACI 371 and ACI 350.3 along with the modified expressions of FEMA 368.

From Table 2 it is seen that as per ACI standards, in velocity-critical range of spectra, the impulsive mode base shear coefficient, C_s decreases as a function of $1/T^{2/3}$. However, in FEMA 368, impulsive base shear coefficient, C_s decreases as $1/T$. For convective mode base shear also similar difference can be noted. To have a better understanding of modifications proposed by FEMA, a comparison of base shear coefficient obtained from ACI 371 expression and one obtained from the modified expression of FEMA368 is shown in Figure 3. The base shear coefficient values shown in Figure 3, correspond to the most sever zone of ACI 371 and equivalent seismic conditions of FEMA 368. It is seen that in short period range ($T < 0.6$ s), base shear coefficient values of FEMA368 are about 12% higher than one obtained from ACI 371. In the long period range, values obtained from both the expressions match well. It may be noted that in the ACI 371, the importance factor does not appear in the expression for base shear coefficient, whereas, FEMA368 modification has introduced an importance factor I =1.25.

Similarly, comparison of base shear coefficient of impulsive mode obtained from the expression of ACI 350.3 and the one modified by FEMA 368 is shown in Figure 4a. These results are for the most severe zone of ACI 350.3 and FEMA 368. It may be noted that in ACI 350.3 value of response modification factor, R_w is in the range of 2.0 to 4.75, whereas in FEMA 368, R varies in the range of 1.5 to 3.0. In Figure 4a, base shear coefficients are shown for the lowest and the highest values of response modification factor. It is seen that for the highest value of response modification factor (i.e. R_w = 4.75 and R = 3.0), base shear coefficient from both the expressions match well. For the lowest value of response modification factor (i.e. R_w = 2.0 and R = 1.5), results of FEMA 368 are on lower side by 15%. In Figure 4b, base shear coefficient corresponding to convective mode is compared. For T > 2.4 s, ACI 350.3 and FEMA 368 expressions give same values of convective base shear coefficient.

3.1 ACI 371 (1998)

This ACI standard provides recommendations for evaluating design seismic forces on concrete pedestal supported elevated tanks. With FEMA modifications, the base shear is given by V = C_s .W, where W is the summation of weight of water, container and support structure above the base. The base shear coefficient C_s is given by

$$C_s = \frac{S_{D1}}{(R/I)T} \quad \text{for } T_s < T < 4 \text{ s} \tag{1}$$

$$C_s = \frac{4S_{D1}}{(R/I)T^2} \quad \text{for } T \geq 4 \text{ s} \tag{2}$$

$$C_s \leq \frac{S_{DS}}{(R/I)} \quad \text{and } C_s \geq 0.2S_{DS} \tag{3}$$

The quantities, S_{DS}, S_{D1}, R, I and T are same as defined in IBC 2000 and T_s = S_{D1}/S_{DS}. FEMA 368 states that except for the above stated modifications, concrete pedestal supported elevated tanks should be designed as per provisions of ACI 371. In this ACI standard, a load factor of 1.1 is given for strength design method, as opposed to unity in IBC 2000. A closer look at the base shear formula mentioned above reveals that, this is same as one given in IBC 2000 except for the minimum values of C_s = 0.2 S_{DS} (which is 0.14 S_{DS} I in IBC 2000) and a load factor of 1.1 for strength design. FEMA 368 specifies R = 2 for pedestal supported elevated tanks. The ratio of base shear coefficient of tank obtained from ACI 371 (i.e. from Eqs. 1 to 3) and base shear coefficient of building (obtained as per IBC 2000 with R = 8) i.e. BSC_{tank} / BSC_{bldg} is plotted in Figure 5. The ratio of tank to building base shear coefficient as obtained from IBC 2000 is also shown in Figure 5. It is seen that values of base shear coefficient obtained from ACI 371 are higher than the one obtained from IBC 2000. The higher value of base shear coefficient as per ACI 371 is due to a load factor of 1.1 and higher value of lower bound limit. ACI 371 does not give details about convective mass component, and recommends its consideration if water weight is less than 80 percent of the total gravity load of tank.

3.2 ACI 350.3 (2001)

ACI 350.3 gives procedure for seismic design of liquid containing concrete tanks, with detailed description of impulsive and convective components. It mainly deals with ground supported concrete tanks and limited information on pedestal supported elevated tanks is also provided. Considering the modifications suggested by FEMA 368, the base shear coefficient for impulsive mode, is given by

$$(C_s)_i = \frac{(0.6\frac{S_{Ds}}{T_0}T_i + 0.4S_{DS})}{1.4(R/I)} \qquad \text{for} \quad 0 < T_i < T_0 \tag{4}$$

$$= \frac{S_{DS}}{1.4(R/I)} \qquad \text{for } T_0 \le T_i < T_s \tag{5}$$

$$= \frac{S_{D1}}{1.4(R/I)T_i} \qquad \text{for } T_i \ge T_s \tag{6}$$

$T_0 = 0.2S_{DS}/S_{D1}$ and T_i is time period of impulsive mode. All other parameters are as defined in earlier sections. Similarly the base shear corresponding to convective component is given by $V_c = (C_s)_c W_c$, where W_c is convective mass and base shear coefficient, $(C_s)_c$ is given by

$$(C_s)_c = \frac{6S_{D1}I}{T_c^2} \qquad \text{for all values of } T_c \tag{7}$$

Here, T_c is the time period of convective mode. In convective mode, damping is taken as 0.5%. It is relevant to note that response modification factor, R does not appear in the expression for convective mode base shear coefficient. This implies that value of R is taken as unity i.e. convective base shear corresponds to pure elastic response.

Variation of base shear coefficient for impulsive and convective modes with time period is shown in Figure 6. Since ACI 350.3 forces correspond to working stress design, they were multiplied by 1.4 to bring the forces to the level of strength design or ultimate load design. It may be noted that FEMA 368 suggests importance factor, I =1.25 for tanks and there is no lower bound limit on base shear coefficient in eqs. 4 to 6. In Figure 7, ratio of base shear coefficient of impulsive mode and base shear coefficient of a building i.e. BSC_{tank} / BSC_{bldg} is plotted. The base shear coefficient of building is obtained as per IBC 2000 with R = 8. It is seen up to T = 2 s, for a tank with R = 1.5, design base shear is 7 times higher than that of a building. However, for longer time period (i.e. T > 2 s), severity of tank shear as compared to that of a building reduces. This is due to the fact that for tanks no lower bound on design seismic force has been defined in ACI 350.3, whereas IBC 2000 specifies a lower bound on design seismic force for buildings.

For convective mode, ACI 350.3 and FEMA 368, specify 0.5% damping and for impulsive mode damping is 5%. Convective mode period is usually grater than 2.0 sec and hence from eq. (6) and (7) it is seen that convective mode spectrum (0.5% damping) is 8.4R/T times higher than impulsive mode spectrum (5% damping).

4. AWWA STANDARDS

American Water Works Association (AWWA) standards provide guidelines for design and manufacturing of different types of water storage tanks. AWWA D-100 (1996) deals with welded steel tanks, AWWA D-103 (1997) is for factory-coated bolted steel tanks. Similarly, AWWA D-110 (1995) deals with wire- and strand- wound, prestressed concrete water tanks and

AWWA D-115 (1995) is for prestressed concrete water tanks with circumferential tendons. All these AWWA standards deal with circular tanks only and quantification of seismic loads provided in them is in a fashion different from IBC 2000. To bring these quantifications in line with IBC 2000, FEMA 368 and its commentary (FEMA 369), has provided modifications to the expressions for base shear coefficients of these AWWA standards. The provisions of these AWWA standards will be discussed along with the modifications of FEMA 368.

4.1 AWWA D-100 (1996) and D-103 (1997)

AWWA D-100 and AWWA D-103 deal respectively with welded steel tanks and factory-coated bolted steel tanks. AWWA D-100 has provisions for ground supported as well as elevated water tanks, whereas AWWA D-103 has provisions for ground-supported tanks only. Provisions of AWWA D-100 and D-103 for seismic design of ground-supported tanks are identical. First the provisions for ground-supported tanks will be discussed.

4.1.1 Ground supported tanks

In Table 3, expressions for base shear coefficients for ground-supported tanks as given in AWWA D-100 and D-103 are given along with the modified expressions of FEMA 368. It is seen that for ground supported steel tanks, both impulsive and convective modes are considered. For impulsive base shear a constant value independent of time period is given. Since ground supported steel tanks will mostly have time period in the constant-acceleration range of spectra, hence it suffices to mention the constant value. It may be noted here that AWWA D-100 specifies the base shear coefficient for working stress design and hence the modified expression given in FEMA 368 contains a factor of 1.4 in the denominator. In AWWA standards, importance factor I is taken as I = 1.25. Further in AWWA D-100, the response reduction factor, R_w varies in the range from 3.5 to 4.5, whereas in FEMA 368, it varies from 2.5 to 3.0. To have better understanding of modifications suggested by FEMA 368, it will be appropriate to compare the numerical values of base shear coefficients obtained from AWWA D-100 and FEMA 368. In the most severe zone of AWWA D-100, for the lowest and highest value of response reduction factor (i.e. R_w = 3.5 and 4.5) the impulsive base shear coefficient turns out to be 0.36 and 0.28 respectively. The corresponding values of base shear coefficient as per the modified expression of FEMA 368, are 0.357 and 0.298 respectively. The parameters of most sever zone of AWWA D-100 are: Z = 0.4, S = 1.5 and soil type C. The equivalent parameters of FEMA 368 are: S_s = 1.5, S_1 = 0.6, F_s = 1.0, F_v = 1.5, S_{DS} = 1.0, S_{D1} = 0.6, T_s = 0.6, site class D.

Regarding the convective mode base shear coefficient it is interesting to observe that in AWWA D-100, convective mode base shear coefficient depends on the response reduction factor. It may be noted that in the provisions of ACI 350.3, the convective base shear is independent of response reduction factor. A comparison of convective base shear coefficient obtained from AWWA D-100 and FEMA 368 for the lowest and highest values of response reduction factor is shown in Figure 8. It is seen that the values obtained from AWWA D-100 expressions are on much higher side. It appears that due to modified expressions of FEMA 368, the convective base shear values have reduced. It is relevant to note here that, as per FEMA 368, the modified expression for convective base shear contains a factor of 1.4 in the denominator to bring the base shear values to working stress level. In this context, one may observe that while suggesting modified expression for convective base shear of ACI 350.3, which is also based on working stress method, FEMA 368 did not use a factor of 1.4 (Table 2). Thus, while modifying the convective base shear expression of AWWA D-100, if a factor of 1.4 is not used in FEMA 368, then the results of AWWA D-100 and FEMA 368 will match well.

4.1.2 Elevated Tanks

For elevated tanks, AWWA D-100 does not consider sloshing effect, and only the impulsive mode is considered. The impulsive base shear is given by V = C_s.W, where, W is the summation of weight of water, container and support structure above the base. The expression for base shear coefficient of elevated tank is given in Table 3, along with the modified expression from FEMA 368. It is seen that as per AWWA D-100, response reduction factor for elevated tank is in the range of 3.0 to 4.0, whereas in FEMA 368 it varies from 2.0 to 3.0. A comparison of the base shear coefficient obtained from AWWA D-100 and FEMA 368 is shown in Figure 9. These results correspond to the most sever zone and are for the lowest and highest values of response reduction factors. In the short period range (i.e., $T < 0.6$ s) base shear coefficient corresponding to the highest response factor (i.e., $R_w = 4$ and $R = 3$) match well. However, for the lowest value of response reduction factor, the FEMA 368 results are on lower side by 15%. In the long period range (i.e., $T > 0.6$ sec), FEMA 368 results are on lower side.

The modified expression for base shear given in FEMA 368, is in the same fashion as that of IBC 2000. In figure 10, the base shear coefficient of elevated tank obtained from the modified expression of FEMA 368, is compared with the base shear coefficient of a ductile building (with $R = 8$, $I = 1.0$ as per IBC 2000). In this figure, the ratio of BSC_{tank}/BSC_{bldg} is plotted for two values of response reduction factor of tanks, i.e., $R = 2$ and 3.0. Since IBC 2000 provisions correspond to strength design, the FEMA 368 values of base shear coefficient are multiplied by 1.4 to bring them to strength design level. It is seen that base shear of elevated tanks is about 3 to 5 times higher than that of a building up to $T = 1.5$ sec. In the long time period range, i.e. $T > 1.5$ sec, the ratio of tank to building base shear reduces which is due to no lower bound limit on tank base shear in the long period range.

4.2 AWWA D-110 (1995)

AWWA D-110 and AWWA D-115 deal with ground supported prestressed concrete tanks, but since their provisions for seismic design are quite different, these two standards will be discussed separately. First, provisions of AWWA D-110 are discussed, and the next section will describe provisions of D-115.

Expressions for impulsive and convective base shear coefficients, as per AWWA D-110 are given in Table 4, along with the modified expressions of

FEMA 368. The base shear coefficients given in AWWA D-110 correspond to working stress method. A comparison of the impulsive base shear coefficient from AWWA D-110 expressions and modified expression of FEMA 368, is shown in Figure 11a. This comparison is shown for the highest and lowest values of response reduction factors. As per AWWA D-110, the values of response reduction factor are in the range of 2.0 to 4.5, whereas in FEMA 368 these values are in the range of 1.5 to 3.0. It is seen that for the highest value of response reduction factor (i.e., R_w = 4.5 and R = 3.0), base shear coefficient values from AWWA D-110 and FEMA 368 match well in the short period range (i.e., T < 0.6 sec). However, for the lowest value of response reduction factor (i.e., R_w = 2.0 and R = 1.5), base shear coefficient values from FEMA 368 is 15% less than that of AWWA D-110. In the long period range (i.e., T > 0.6 sec), base shear coefficient values obtained from FEMA 368 are on lower side for the highest as well as lowest response reduction factor.

Regarding the modified convective base shear expressions of FEMA 368, there are certain inconsistencies. As per AWWA D-110, convective base shear coefficient does not depend on response reduction factor (i.e., R_c = 1.0 for all types of tanks). In section 14.7.3.7.3 of FEMA 368, on page no. 311 it is mentioned that the modified convective base shear coefficient shall be taken as $(C_s)_c = \frac{6S_{D1}I}{T_c^2}$. However, on page no. 312 it is stated that the modified convective base shear is given by $V_c = \left(\frac{6S_{D1}I}{1.4RT_c^2}\right)W_c$. Clearly there is inconsistency in the two expressions, second one considers a factor of 1.4R in the denominator and the first one does not. Further, in FEMA 369 (pp 366), which is commentary to FEMA 368, the modified base shear expression is also given as $(C_s)_c = \frac{6S_{D1}I}{T_c^2}$. Since in AWWA D-110 the convective base shear coefficient does not depend on response reduction factor, in Table 4, the modified base shear coefficient is taken as $(C_s)_c = \frac{6S_{D1}I}{T_c^2}$. A comparison of convective base shear coefficient obtained from AWWA D-110 and FEMA 368 is shown in Figure 11b. It is seen that base shear coefficient values obtained from FEMA 368 about 1.5 times higher than those obtained from AWWA D-110.

Impulsive base shear coefficients given by the modified expressions of FEMA 368 are in line with those given by IBC 2000 for buildings. Hence these expressions of FEMA 368 can be used to compare the base shear coefficient of tank with that of a ductile building (R = 8 and I =1 as per IBC 2000). Such a comparison is shown in Figure 12, wherein, ratio of BSC_{tank}/BSC_{bldg} is plotted for three different values of response reduction factor of tanks. While plotting these results, the base shear coefficient of tank is multiplied by 1.4 to bring them to strength design level. Again, due to absence of a lower bound limit on base shear coefficient of tanks, the values of BSC_{tank}/BSC_{bldg} decreases in long period range, i.e., for T > 2.0 sec.

4.3 AWWA D-115 (1995)

This AWWA standard deals with circular prestressed concrete tanks with circumferential tendons. Expressions for impulsive and convective base shear coefficients are given in Table 5 along with the modified expressions of FEMA 368. It will be appropriate to note the differences in the base shear coefficient expressions of AWWA D-110 and AWWA D-115. In AWWA D-115, the response reduction factor varies from 1.0 to 3.0 whereas in AWWA D-110 it varies from 2.0 to 4.75.

A comparison of impulsive base shear coefficient obtained from AWWA D-115 and modified expression of FEMA 368 is shown in Figure 13, for extreme values of response reduction factors. It is surprising to observe a large difference in the base shear coefficient values obtained from these two expressions. The values obtained from FEMA 368 expressions are on lower side and there appears to be some inconsistency in the expressions of AWWA D-115 and the modified expression of FEM 368. In this context, it is to be noted that FEMA 368 suggests same modified expression for impulsive base shear coefficient of AWWA D-110 as well as AWWA D-115. Thus as per FEMA 368, the values of response reduction factor are identical for AWWA D-110 as well as AWWA D-115. However, as pointed out earlier, response reduction factor values in AWWA D-110 and AWWA D-115 are quite different. Thus, it appears that while suggesting the modified base shear expressions, FEMA 368 has not considered this difference in the range of values of response reduction factors of AWWA D-110 and D-115.

Another major inconsistency noted is that in AWWA D-115 convective base shear coefficient depends on response reduction factor, whereas in FEMA 368 the corresponding modified expression does not have any dependence on response reduction factor. Due to this, it is not possible to compare the convective base shear coefficient values obtained from AWWA D-115 and FEMA 368. Again it may be noted that the FEMA 368 expression for convective base shear coefficient is same for AWWA D-110 and D-115, and in AWWA D-110 also convective base shear does not depend on response reduction factor.

It is quite clear that due to the inconsistencies mentioned above, no comparison can be done between the impulsive base shear coefficient of tank obtained from modified expression from FEMA 368 and the base shear coefficient of building.

Thus, a review of provisions of AWWA standards and their corresponding modifications in FEMA 368 reveals quite a few inconsistencies and contradictions. These are summarized below:

(a) In AWWA D-100 and D-115 convective base shear coefficient depends on response reduction factor whereas, in AWWA D-110 this coefficient does not depend on response reduction factor.

(b) Though AWWA D-110 and D-115 deal with prestressed concrete tanks, they use quite different values of response reduction factors. Moreover, in the modified expression of FEMA 368, only one range of value of response reduction factor is suggested for AWWA D-110 and D-115. This led to a large difference in the values of base shear coefficients obtained from AWWA D-115 and FEMA 368.

(c) In AWWA D-115, convective base shear depends on response reduction factor, however, the base shear given by the corresponding modified expression of FEMA 368 does not depend on response reduction factor.

(d) All AWWA codes are based on working stress method. While modifying the convective base shear coefficient of AWWA D-100, the FEMA 368 has considered a factor of 1.4 in the expression; however this factor is not present in the modifications suggested for AWWA D-110 and D-115. In this context it may be noted that ACI 350.3 is also based on working stress method, and while modifying its convective base shear expression, FEMA 368 has not considered a factor of 1.4 in the expression (Table 2).

5. API 650 (1998)

American Petroleum Institute (API) has two standards namely, API 650 and API 620 which provide provisions for design and construction of petroleum steel tanks. These API standards deal with ground supported circular steel tanks only. The seismic design provisions in these API standards are identical and hence provisions of only API 650 are described here. API 650 considers impulsive as well as convective component in the seismic analysis. The base shear corresponding to impulsive and convective components is respectively given by $V_i = C_i W_i$ and $V_c = C_c W_c$. Here, W_i and W_c are impulsive and convective masses respectively and likewise, C_i and C_c are base shear coefficients corresponding to these components. In API 650, quantification of base shear coefficients is in a manner different than IBC 2000. Hence, FEMA 368 has suggested modifications to base shear coefficients of API 650, to make them consistent with IBC 2000. Table 6 provides details of base shear coefficients given in API 650 along with the modified expressions given by FEM 368

For impulsive mode, the base shear coefficient is defined as a constant value which does not depend on the time period. It may be recognized here that ground supported steel tanks will generally have time period in the constant-acceleration range of spectra and hence it suffices to specify the magnitude of the spectra in constant-acceleration range. It may be recalled here that AWWA D-100 has also specified a constant value of base shear coefficient for ground supported steel tanks. In API 650 and FEMA 368 the value of importance factor is taken as 1.25. API 650 prescribes seismic forces at working stress level, and they are to be multiplied by a factor of 1.4, to bring them strength design level. Thus, as per API 650, C_i = 0.84 ZI and as per FEMA 368, at strength design level, $C_i = 0.336 S_{DS} I$, which when compared with IBC 2000 expression (i.e. $S_{DS}I/R$), implies that for ground supported steel tanks, R = 2.976 ≈ 3.

For the most severe zone of API 650, Z = 0.4 and for S3 type site, S = 1.5. For equivalent seismic condition, parameters from FEMA 368 will be S_D = 1.5, F_a = 1.0, S_1 = 0.6 and F_v = 1.5, S_{DS} = 1.0, with site class D. For these parameters, the value of C_i, at strength design level, will be 0.42 from API 650 as well as FEMA 368. This value of base shear coefficient is about 3.4 times higher than the value of base shear coefficient of a building (obtained as per IBC 2000 with R = 8). Comparison of convective mode base shear coefficient, at strength design level, obtained from API 650 and FEMA 368 is shown in Figure 14. It is seen that FEMA 368 results are about 20% less than those of API 650.

6. EUROCODE 8 (1998)

Five parts of Eurocode 8 (1998) cover provisions for seismic design of various types of civil engineering structures. Part-4 of Eurocode 8 deals with tanks, silos and pipelines. This code describes in detail about dynamic modeling of convective and impulsive components, and also discusses other aspects like base uplifting of unanchored tanks, soil-structure interaction etc.

The seismic action is specified in terms of response spectrum. In this code, behavior factor, q, accounts for energy dissipation capacity of the structure, mainly through ductile behavior and other mechanisms. For elastic structures, q = 1.0 and for structures with good energy dissipation capacity, q = 5.0. Eurocode 8 specifies two types of response spectrum, first one is *elastic spectrum*, $S_e(T)$, (for q =1.0) and second is *spectrum for linear analysis*, $S_d(T)$, (for q > 1.0). For tanks on ground, *elastic spectrum* is to be used, i.e., behavior factor, q = 1.0. For buildings with ductile frames, behavior factor q can be as high as q = 5.0, and *spectrum for linear analysis* is used. Seismic base shear for building is given by V = γ_I $S_d(T)$ W, where, γ_I is the importance factor and $S_d(T)$ is the spectrum acceleration at time period T. Expression for base shear of tank is same as that of building, except that instead of *spectrum for linear analysis* $S_d(T)$, *elastic spectrum* $S_e(T)$ is to be used. The expressions for elastic spectrum and spectrum for linear analysis are given in Table 7. A closer look at these expressions reveals that *elastic spectrum* depends on damping correction factor η, whereas, *spectrum for linear analysis* does not depend on η. Further, it can be seen that there is a lower bound limit on values of *spectrum for linear analysis* in long period range. There is no such lower bound limit on *elastic spectrum*.

It may be noted that Eurocode 8 provides indicative values of behavior factor, q and various parameters defining shape of spectrum. However, there is no indication on maximum value of design ground acceleration, a_g, which corresponds to a reference return period of 475 years. It mentions that National Authorities of the member countries should arrive at suitable values of a_g for various seismic zones. In the present study, value of a_g = 0.3g (i.e., α = 0.3) is assumed.

Figure 15 shows variation of base shear coefficient with time period for ductile building (q= 5.0) and impulsive mode of tank. These results correspond to S =1.0, β = 2.5, η= 1.0, K_1 =1.0, K_2 =2.0, K_{d1} =2/3, K_{d2} =5/3, T_B =0.15, T_C =0.6 T_D = 3.0 and α = 0.3. For buildings, γ_I =1.0 and γ_I =1.2 for tanks with high risk to life, and large environmental, economic and social consequences. Sub soil class B is considered which is similar to site class D of IBC 2000. In Figure 15, base shear coefficient for convective mode with 0.5% damping is also shown. It is to be noted that for buildings, i.e., when *spectrum for linear analysis* is used, there is a lower bound limit on spectrum values, however for *elastic spectrum* no such limit is specified. Further, it is important to note that shapes of $S_e(T)$ and $S_d(T)$ are also different beyond T = T_B. *Elastic spectrum*, $S_e(T)$ reduces much faster with time period than *spectrum for linear analysis*, $S_d(T)$. Hence, for higher time periods (T>0.6 s), the difference between base shear of tank and building is reduced considerably. This is clearly seen from Figure 16, wherein ratio BSC_{tank} / BSC_{bldg} is plotted. Eurocode 8 specifies only one value of q (= 1.0) for ground-supported tanks. Neither it mentions about different types of ground-supported tanks nor does it give specific values of q for different types of ground-supported tanks. However, it states that if adequately demonstrated, inelastic response (i.e., q > 1) can be considered.

For elevated tanks also, Eurocode 8 does not give very specific values of q. It mentions that supporting structure may be designed to respond beyond the yield level, thereby allowing energy dissipation in it. Elevated tanks with simple support and which have little risk to life, negligible environmental and social consequences due to failure, will have the value of q = 2.0. For elevated tanks under higher risk category, the selected value of q should be properly substantiated and proper ductility be provided through ductile design of supporting structure.

For convective mode, in all types of tanks, the value of behavior factor, q =1.0 and damping value, ξ = 0.5% is suggested. For this value of ξ, the damping correction factor η is 1.673. As a result of this, 'elastic spectrum' corresponding to convective mode (0.5% damping) turns out to be 1.673 times higher than that for impulsive mode (5% damping).

7. NZSEE GUIDELINES AND NZS 4203 :1992

In New Zealand, seismic design of liquid storage tanks follow NZSEE's document (Priestley, *et. al.,* 1986). A study group of NZSEE is presently revising this document to bring it in line with the NZS 4203:1992, the New Zealand loading code for buildings. Details of the proposed seismic loading are given by Whittaker and Jury (2000). This section discusses the proposed seismic loading along with the seismic loading provisions for building in NZS 4302:1992.

As per NZS 4203:1992, seismic base shear for building is given by V = CW, where, lateral force coefficient or base shear coefficient, C is given by

$$C = C_h(T, \mu)\, S_p\, R\, Z\, L_u \tag{7}$$
$$\geq 0.03$$

where

$C_h(T, \mu)$ = Basic seismic hazard coefficient which accounts for different soil conditions, ductility factor μ and natural period T of building,

S_p = Structural performance factor. For buildings $S_p = 0.67$

R = Risk factor. For ordinary buildings R =1.0

Z = Zone factor as per seismic map

L_u = Limit state factor for ultimate limit state. $L_u = 1.0$

NZS 4203:1992 has provided tabulated values of $C_h(T, \mu)$ for three different soil conditions, eight different values of μ (from $\mu = 1.0$ to 10.0) and values of time period T from T = 0.0 to 4.0 sec. For buildings with ductile frames the value of μ could vary from $\mu = 6$ to10.

As per Whittaker and Jury (2000), seismic base shear for tanks is V=CW, where

$$C = C_h(T, 1)\, S_p\, R\, Z\, L_u\,)\, C_f(\mu, \xi\,) \tag{8}$$

Eq.(8) differs from Eq.(7) in two ways: First, basic seismic hazard coefficient, $C_h(T,1)$ corresponds to μ =1.0, i.e., purely elastic spectrum is used and secondly, an additional factor, $C_f(\mu,\xi)$ termed as correction factor is included. This correction factor accounts for ductility and level of damping. For tanks, performance factor S_p=1.0 is recommended as opposed to 0.67 for buildings. Value of risk factor, R for tanks is arrived at by considering four aspects, namely, risk to number of persons, risk to environment, community significance of the tank and value of adjacent property. Value of R can vary from 0.5 to 1.6 depending on the risk associated with the tank and a tank with serious risk has R = 1.3. The value of damping depends on material of tank shell and supporting soil. Whittaker and Jury (2000) have provided values of ductility factor, μ for different types of tanks, which are shown in Table 8. Similarly, damping level, ξ for different types of concrete and steel tanks are also provided. Further, provisions are made for increasing the damping values of tank depending on flexibility of supporting soil, i.e., to consider radiation damping in soil. For different values of ductility factor, μ and damping level, ξ, values of correction factor C_f (μ, ξ) are provided as shown in Table 9.

For three different values of C_f, namely, C_f=0.72, 0.54 and 0.38, variation of base shear coefficient with time period is plotted in Figure 17. These results are for the most severe zone (Z =1.2) and site subsoil category (C), i.e., flexible and deep soil condition, which is similar to site class D of IBC 2000. Also plotted in this figure is the variation of base shear coefficient for a building with μ = 6.0. It would be appropriate to note that variation of C_h(T,6) and C_h(T,1) with time period is of different nature. This gets reflected in the variation of base shear coefficient of tanks and building with time period. For T > 0.6s, reduction in C_h(T,1) with time period is slower than corresponding reduction in C_h(T,6). This implies that elastic spectrum (C_h(T,1)) is more flatter than inelastic spectrum (C_h(T,6)). Moreover values of C_h(T, 1) and C_h(T,6) at various time period do not have a constant ratio.

From the ratio of BSC_{tank} / BSC_{bldg} shown in Figure 18, it is seen that the ratio of base shear coefficients increases at around T= 0.6s, then remains almost constant up to T=3.0s and for T>3.0s it starts decreasing. The increase is due to the fact that values of C_h(T, 6) (i.e. for building) and C_h(T, 1) (i.e., for tank) are not in same proportion for values of T from 0.6-3.0s. The decrease in ratio of base shear for higher value of T (i.e., T > 3.0s) is due to the fact that there is a lower bound on value of C (i.e., C can't be less than 0.03) for buildings, but for tanks (i.e., elastic case of μ =1.0) there is no such lower bound limit.

For elevated tanks, Whittaker and Jury (2000) do not provide specific information on ductility factor, μ. However, it mentions that for elevated tanks, ductility factor as appropriate for support structure should be considered. This may imply that if supporting structure is quite ductile then value of μ can be as high as for buildings (i.e., μ = 6 to 10). At the same time it should be noted that in Table 9, values of response modification factor, C_f is given only for maximum value of μ = 4.0, i.e., for values of μ greater than 4.0, response modification factor is not available.

Whittaker and Jury (2000) have specified 0.5% damping for convective mode. Thus, in Table 9, values of $C_f(\mu,\xi)$ corresponding to ξ = 0.5, can be used for convective mode. These values of $C_f(\mu,\xi)$ change with ductility factor, μ, implying that convective mode base shear will vary with ductility of tank. It may be noted that in AWWA D-100 also convective base shear depends on ductility of tank i.e. on response reduction factor. However, in ACI 350.3 and Eurocode 8, convective mode shear does not depend on ductility of tank and it corresponds to pure elastic case, i.e., R = 1, and q =1.0, respectively.

8. COMPARISON OF DESIGN FORCES FROM VARIOUS CODES

In the previous sections provisions related to base shear coefficients for tanks as given in various codes, standards and guidelines are described. It will be interesting to compare base shear coefficients obtained from these documents. Before comparing the results for tanks, a comparison of base shear coefficient for a ductile building is shown in Figure 19. In this figure, base shear coefficients of building (BSC_{bldg}), obtained from IBC 2000, Eurocode 8 and NZS 4203:1992 are shown. These results correspond to the most severe zone of each code. It is seen that in the short period range (i.e., T=0.1-0.6s), results from Eurocode 8 and NZS 4203 match well. In this short period range, IBC 2000 results are on lower side by about 15%. Further, all the three codes have different shape of spectra in constant-velocity range (i.e., T>0.6s). Moreover, magnitude of the lower bound limit on spectra is also seen to be different in these codes. To obtain similar comparison for tanks, first of all, for a particular type of tank, all the relevant parameters (such as R, q, C_f) from different codes will have to be identified. It is seen that most of the codes consider ground supported unanchored concrete water tank as a low ductility tank or a tank with low energy absorbing capacity. For such a tank the relevant parameters will be as shown in Table 10. In Figure 20, comparison of base shear coefficient for this tank (BSC_{tank}) obtained from different codes is shown. It may be noted here that FEMA 368 has modified the base shear expressions of ACI 350.3 and AWWA D-110 and brought them in line with IBC 2000. In view of these modifications, parameters from ACI 350.3, AWWA D-110 and IBC 2000 are same. From Figure 20 it is seen that in the short period range ($T<0.6s$), Eurocode 8 results are 10% higher and NZSEE results are 35% higher than the one obtained from IBC2000. Further, it can also be seen that except for IBC 2000, no other code has lower bound limit on base shear coefficient in long period range. Comparison of ratio of base shear coefficient of tank and building (BSC_{tank}/BSC_{bldg}) is shown in Figure 21. Here, base shear coefficient of tank from a particular code is divided by corresponding base shear coefficient of a ductile building. It is seen that from T=0.1-0.6s, this ratio is constant for all the codes. This constant value is 6 for Eurocode 8 and for IBC and NZSEE it is 6.7 and 7.3 respectively. The decrease in the value of this ratio for $T>0.6s$ for the case of Eurocode 8, is due to difference in shapes of spectrum used for tank and building. Another factor contributing to this decrease, particularly in higher period range, is absence of lower bound limit on spectral values for tanks. The decrease in the value of this ratio in long period range, for NZSEE, ACI 350.3 and AWWA D-110 is also attributed to similar reasons. For the case of IBC 2000, due to lower bound limit on spectral values for tanks, the ratio of tank to building shear does not fall below the value of 4, even in long period range.

Results similar to one presented in Figure 21, can be obtained for a high ductility tank, i.e., a tank with high energy absorbing capacity. For such a tank, various parameters of different codes are given in Table 11. These parameters can as well be applicable to some of the elevated tanks. For Eurocode 8, value of $q = 2$ is considered, which is suggested for a low risk category elevated tank with simple type of supporting structure. Results on ratio of base shear coefficient of tank to building, (BSC_{tank} / BSC_{bldg}), are shown in Figure 22. It is seen that maximum value of this ratio is about 3 to 4 in all the codes, as against a value of 6 to 7 for low ductility tanks. This implies that design base shear for a low ductility tank is double that of a high ductility tank. Variation in the ratio of base shear of tank and building, in the higher time period range is seen in Figure 22 also, which is due to reasons discussed earlier.

On similar lines, comparison of convective base shear coefficients obtained from various codes is shown in Figure 23. Before looking into this results it will be appropriate to recognize that as per AWWA D-100, AWWA D-115 and NZSEE (i.e., Whittaker and Jury (2000)), the convective base shear coefficient depends on response reduction factor or ductility factor. However, as per ACI 350.3, AWWA D-110, API 650 and Eurocode 8 the convective base shear coefficient does not depend on response reduction factor. The comparison shown in Figure 23 considers only those documents in which convective base shear coefficient does not depend on response reduction factor. Hence in Figure 23, results of ACI 350.3, AWWA D-110, API 650 and Eurocode 8 are shown. It can be seen that convective base shear coefficient obtained from Eurocode 8 and API 650 match well. However, ACI 350.3 gives very high values of convective base shear coefficient. For time period greater than 3.0s, the results of ACI 350.3 are about 2.5 times higher than that of API 650.

9. PROVISIONS OF INDIAN CODE

Indian Standard IS:1893-1984 provides guidelines for earthquake resistant design of several types of structures including liquid storage tanks. This standard is under revision and in the revised form it has been divided into five parts. First part, IS 1893 (Part 1):2002, which deals with general guidelines and provisions for buildings has already been published. Second part, yet to be published, will deal with the provisions for liquid storage tanks. In this section, provisions of IS:1893-1984 for buildings and tanks are reviewed briefly followed by an outline of the changes made in IS 1893 (Part 1):2002.

In IS: 1893-1984, base shear for building is given by $V = C_sW$, where, C_s is the base shear coefficient given by

$$C_s = K\, C\, \beta I\, \alpha_o \qquad (9)$$

Here, K = Performance factor depending on the structural framing system and brittleness or ductility of construction; C = Coefficient defining flexibility of structure depending on natural period T; β = Coefficient depending upon the soil-foundation system; I = Importance factor; α_o = Basic seismic coefficient depending on zone. For buildings with moment resisting frames, K = 1.0. Importance factor, for buildings is usually I = 1.0.

IS:1893-1984, does not have any provision for ground supported tanks. It has provisions for elevated tanks, for which it does not consider convective mode. Base shear for elevated tank is given by $V = C_s W$, where, base shear coefficient, C_s is given by

$$C_s = \beta I F_o (S_a/g) \tag{10}$$

Here, S_a/g = average acceleration coefficient corresponding to the time period of the tank, obtained from acceleration spectra given in the code; F_o = seismic zone factor; W = weight of container along with its content and one third weight of supporting structure. For elevated tanks, Importance factor I = 1.5. It may be noted that in the expression for base shear coefficient of tank, the performance factor K does not appear, i.e. K = 1 is considered, which is same as that for a building with ductile frame. This implies that in IS:1893-1984, there is no provision to account for lower ductility and energy absorbing capacity of elevated tanks. Thus, as per IS:1893-1984, base shear coefficient for tank will be only 1.5 times higher than that for a building, which is due to higher value of importance factor. This is in contrast to other codes, reviewed in earlier sections, wherein tank base shear coefficient is seen to be 3 to 7 times higher than buildings. This lacunae needs to be corrected in the next revision of the code.

As mentioned earlier, IS 1893 is under revision and first part, of the revised code, IS 1893 (Part 1): 2002, has already been published. In this revised code, base shear for building is given by $V = C_s W$, and base shear coefficient C_s is given by

$$C_s = \frac{ZI}{2R}(S_a/g) \tag{12}$$

where Z = zone factor, I = importance factor, R = response reduction factor and S_a/g = average response acceleration coefficient, obtained from acceleration spectra given in the code. For buildings with ductile frames value of R is 5. In Figure 24, a comparison of base shear coefficients for building obtained from

IS:1893-1984 and IS 1893 (Part I):2002 is shown, along with the base shear coefficient from IBC 2000. Since IS:1893-1984, does not specify specific value of load factors for strength design, the results in Figure 24 are presented for working stress level. It is seen that base shear coefficient from IS:1893-1984 is lower than one from IS 1893 (Part I):2002. Further, unlike IBC 2000, there is no lower bound in IS 1893 (Part I):2002 and IS:1893-1984.

Subsequent parts of IS 1893:2002, will be using acceleration spectra given in Part I, and will be based on same design philosophy. Thus, for liquid storage tanks, base shear coefficient will be given by Eq.(12), in which suitable values of R will have to be used for different types of tanks. From the review presented in earlier sections, it is seen that low and high ductility tanks have design base shear 3 to 7 times higher than ductile buildings. In Figure 25, base shear coefficients for low and high ductility tanks, from IBC 2000 (i.e., tanks with R=1.5 and R=3.0) are shown. To achieve this level of base shear coefficients the value of R in IS 1893 (Part 1):2002 should be 1.1 and 2.25 as can be seen from Figure 25. Also shown in this figure is the base shear coefficient for tank obtained from IS:1893-1984, which is on much lower side. Based on the comparison shown in Figure 25, proposed values of R which can be used in IS 1893 (Part 2):2002 for different types of tanks, are given in Table 12.

10. DISCUSSION

Due to low ductility and energy absorbing capacity, liquid storage tanks are generally designed for higher seismic forces as compared to conventional buildings. In this article, provisions of various codes on design seismic forces for tanks are reviewed.

It is found that there is considerable variation in the types of tanks described in various codes. For example, ground supported tanks described in IBC 2000, ACI 350.3, AWWA D-110 and NZSEE guidelines are having different types of base conditions. Eurocode 8 does not provide any details about base supports of ground-supported tanks. Provisions on elevated tanks are described in IBC 200, ACI 371, ACI 30.3, AWWA D-100, Eurocode 8 and

NZSEE guidelines. Less information is available on energy absorbing capacity of different types of supporting towers of elevated tanks. Less frequent use of elevated tanks in these countries may be one reason for low emphasis on elevated tanks in these codes.

All the codes, consider impulsive and convective modes of vibration in the seismic analysis of ground-supported tanks. The level of design seismic force for a particular tank obviously depends on its ductility and energy absorbing capacity. It is observed that for a tank with low ductility, impulsive base shear coefficient (ratio of lateral force to weight) is 6 to 7 times higher than the base shear coefficient of a ductile building; and for a high ductility tank this value is 3 to 4 in all the codes (Figures 21 and 22). However, this is so only for tanks with short time period (i.e., T<0.6s). Beyond this short period range, there is considerable difference in the values of $BSC_{tank}/BSC_{bldg.}$ For example, at T=1.5s, for a tank with low ductility, the value of BSC_{tank}/BSC_{bldg}, as per NZSEE guideline is 8.2 and as per IBC 2000 and Eurocode 8 this value is 6.7 and 4.4, respectively. Thus, Eurocode 8 results are on lower side by almost 50%. In fact, beyond T=0.6s, as per Eurocode 8, the value of BSC_{tank}/BSC_{bldg}, decreases continuously, which is due to two reasons. First, in Eurocode 8, *elastic spectrum* (used for tank) has much faster reduction with time period than *spectrum for linear analysis* (used for buildings) in the constant-velocity range. Secondly, unlike for buildings, there is no lower bound limit on spectrum used for tanks.

In NZSEE guidelines [i.e., NZS 4230:1992 and Whittaker and Jury (2000)] the *elastic spectrum* (used for tanks) reduction with time period is slower than *inelastic spectrum* (used for buildings). Due to this reason, the NZSEE results in Figure 21, show a slight increase in the value of BSC_{tank}/BSC_{bldg} at T=0.6s. In IBC 2000, spectra used for tank and building, have same shape in constant-velocity range. Further, IBC 2000 specifies, lower bound limits on spectral values for buildings as well as tanks. Hence, as per IBC 2000, values of BSC_{tank}/BSC_{bldg}, do not fall below four even in the long period range. In case of ACI 350.3 and AWWA D-110 also, there is no lower bound specified on spectral values for tanks. This, as explained earlier, leads to loss of severity of tanks shear in long period range as compared to that of buildings (Figures 21 and 22).

The reason for providing a lower value of response reduction factor for tanks is their low ductility and low energy absorbing capacity as compared to buildings. However, it is seen that in many codes, due to non-availability of lower bound limit on spectral values of tanks, the ratio of tank to building base shear reduces in long period range. ACI 371, which deals with shaft supported elevated tanks, is a good example on provision of lower bound limit on spectral values. Due to availability of suitable lower bound limit in ACI 371, the value of BSC_{tank} / BSC_{bldg} does not reduce even in long period range (Figure 5). Thus, there is a need to provide suitable lower bound limit on spectral values of tanks in other codes. Absence of such a lower bound limit can lead to non-conservative estimate of base shear for tanks with longer time period, particularly the elevated tanks on flexible supports.

For the case of ground supported steel tanks, API 650 and AWWA D-100 specify a constant value of base shear coefficient, which does not depend on time period. For steel tanks, it is quite likely that impulsive time period will be in the constant-acceleration range of spectra, and hence it suffices to specify this constant value of base shear coefficient. However, in IBC 2000, the base shear coefficient for ground supported steel tanks is not defined as a constant value. Further, as per AWWA D-100 and IBC 2000, response reduction factor for ground supported steel tanks changes with type of base support provided. However, in API 650, tanks with different types of base supports are not described. As per API 650, the base shear coefficient for ground supported steel tank is about 3.4 times higher than that of a ductile building; and in AWWA D-100 and IBC 2000, depending on type of base support, the base shear coefficient of ground supported steel tank is 3 to 3.7 times higher.

While considering the convective base shear, all the codes suggest a damping value of 0.5%. However, in the evaluation of convective base shear coefficient, considerable differences are seen in the provisions of various codes.

Firstly, as per ACI 350.3, API 650, AWWA D-110 and Eurocode 8, convective base shear coefficient does not depend on response reduction factor. However, as per AWWA D-100, AWWA D-115 and NZSEE guidelines (i.e. Whittaker and Jury (2000)), convective base shear coefficient depends on response reduction factor. From the comparison presented in Figure 23, it is seen that base shear coefficient evaluated as per ACI 350.3 is about two and half times greater than the one obtained from AWWA D-110 and Eurocode 8.

For the elevated tanks, AWWA D-100 does not recommend consideration of convective mode of vibration. However, Eurocode 8 and NZSEE guidelines recommend consideration of convective mode. At the same time, IBC 2000 and ACI 371 suggest that convective mode need not be considered if certain conditions on weight of water and time period of convective mode are met with.

As far as AWWA Standards are considered, there appears to be quite a few inconsistencies in them and also in the modifications suggested for them in FEM 368. These have been described in detail in section 4. There is a need to properly address these inconsistencies.

In the context of Indian codes it is noted that design seismic forces for buildings, as per revised Indian code (i.e., IS 1893 (Part 1):2002), compare well with those specified in IBC 2000 (Figure 24). However, Indian code does not have a lower bound limit on spectral values for buildings, which otherwise is present in all the other codes. As far as liquid storage tanks are concerned, Indian scenario is bit different. In India, elevated tanks are quite commonly used in public water distribution systems and a large number of them are in use. These tanks have various types of support structures, like, RC braced frame, steel frame, RC shaft, and even masonry pedestal. Ground supported tanks are used mainly by petroleum and other industrial installations. For different types of elevated and base supports for ground-supported tanks, values of response modification factor, R, to be used in Indian code are proposed (Table 12). However, it is felt that for elevated tanks with different types of supporting structures, a detailed investigation is needed to ascertain their energy absorbing capacity and ductility characteristics. Similarly, suitable values of lower bound limits on spectral values for buildings as well as other types of structures, including tanks, needs to be arrived at.

11. CONCLUSIONS

Following conclusions are drawn from the comparative assessment of provisions of different codes on seismic design of liquid storage tanks:

1) There is no uniformity in types of tanks described in various documents. Most of the codes put emphasis on ground-supported tanks and very limited information is available on elevated tanks.
2) All the documents suggest consideration of convective and impulsive components in seismic analysis of tanks.
3) For a particular type of tank with short period (less than 0.6s), ratio of base shear of tank and building is almost same in all the codes. This ratio is 6 to 7 for low ductility tanks and 3 to 4 for high ductility tanks. However, for tanks with time period greater than 0.6s, there is a large variation in the values of this ratio obtained from different codes. For example, at time period of 1.5 sec, value of this ratio from Eurocode 8 is almost 50% less than the one obtained from NZSEE guidelines. This is attributed to the use of spectra of different shapes for buildings and tanks.
4) Unlike for buildings, most of the documents do not provide lower bound limit on spectral values for tanks. This results in decrease in the ratio of base shear of tank and building, in long period range. This effectively results in reduction in severity of tank base shear as compared to building base shear.
5) Suitable provisions for lower bound limit on spectral values for tanks are necessary. Only ACI 371, which deals with elevated tanks, and IBC 2000 have provisions for lower bound limit on spectral values of tanks.
6) Convective mode base shear values obtained from API 650 and Eurocode 8 match well, however one obtained as per ACI 350.3 is 2.5 times higher than that of ACI 350.3.

7) There are quite a few inconsistencies among different AWWA standards. These need to be resolved. Further modifications suggested by NEHRP recommendations (FEMA 368) to provisions of AWWA codes are also having certain inconsistencies.
8) Indian code needs to include provisions on lower bound limit on spectral values of buildings and tanks. Further, provisions for inclusion of convective mode of vibration in the seismic analysis of tanks also need to be included. Based on the review of various international codes presented in this paper, it is recommended that IS 1893 should have values of response reduction factor in the range of 1.1 to 2.25 for different types of tanks.

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Table 1: Various types of tanks and their R values as mentioned in IBC 2000

Type of tank	R
Ground supported tanks	
Flat bottomed welded or bolted steel tanks – Anchored	3.0
Flat bottomed welded or bolted steel tanks – Unanchored	2.5
Reinforced or prestressed concrete tanks with anchored flexible base	3.0
Reinforced or prestressed concrete tanks with reinforced nonsliding base	2.0
Tanks with unanchored and unconstrained flexible base	1.5
<u>Elevated tanks</u>	
On braced legs	3.0
On unbraced legs	3.0
On irregular braced legs single pedestal or skirt supported	2.0
Tanks supported on structural towers similar to buildings	3.0

Table 2: Expressions for base shear coefficients from ACI and FEMA 368

ACI 371	**FEMA 368**
$C_s = \frac{1.2C_V}{RT^{2/3}}$ $\leq \frac{2.5Ca}{R}$ $\geq 0.5\ C_a$ Values of coefficients, C_V and C_a depend on seismic zones; R is response modification factor with maximum value of 2.0	$C_s = \frac{S_{D1}}{(R/I)T}$ for $T_s < T < 4$ s $= \frac{4S_{D1}}{(R/I)T^2}$ for $T \geq 4$ s $\leq \frac{S_{DS}}{(R/I)}$ $\geq 0.2\ S_{DS}$ R = 2.0 for pedestal supported elevated tanks
ACI 350.3	**FEMA 368**
Impulsive mode $(C_s)_i = \frac{2.75ZI}{R_W}$ for $T_i \leq 0.31$ s $= \frac{1.25ZIS}{R_W T_i^{2/3}} < \frac{2.75ZI}{R_W}$ for $T_i > 0.31$ s **Convective mode** $(C_s)_c = \frac{1.875ZIS}{T_c^{2/3}}$ for $T_c < 2.4$ sec $< 2.75\ ZI$ $= \frac{6ZIS}{T_c^2}$ for $T_c \geq 2.4$ sec Z is zone factor; S is soil profile coefficient; I is importance factor and R_W is response modification factor. Values of R_W are in the range of 4.75 to 2.0 for different types of tanks.	**Impulsive mode** $(C_s)_i = \frac{(0.6\frac{S_{Ds}}{T_0}T_i + 0.4S_{DS})}{1.4(R/I)}$ for $0 < T_i < T_0$ $= \frac{S_{DS}}{1.4(R/I)}$ for $T_0 \leq T_i < T_s$ $= \frac{S_{D1}}{1.4(R/I)T_i}$ for $T_i \geq T_s$ **Convective mode** $(C_s)_c = \frac{6S_{D1}I}{T_c^2}$ for all values of T_c. Values of R are in the range of 1.5 to 3.0 for different types of tanks

Table 3: Expressions for base shear coefficients from AWWA D-100, D-103 and FEMA 368

AWWA D 100 and D103 *(Ground supported tanks)*	**FEMA 368**
Impulsive mode $(C_s)_i = \frac{2.52ZI}{R_w}$ **Convective mode** $(C_s)_c = \frac{3ZIS}{R_w T_c}$ for $T_c \le 4.5$ s $= \frac{13.5ZIS}{R_w T_c^2}$ for $T_c > 4.5$ s Z is zone factor; I is importance factor; S is soil factor; T_c is time period of convective mode; R_w is response reduction factor with values in the range of 3.5 to 4.5	**Impulsive mode** $(C_s)_i = \frac{S_{DS} I}{1.4R}$ **Convective mode** $(C_s)_c = \frac{1.5 S_{D1} I}{1.4 R T_c}$ for $T_s < T_c \le 4$ s $= \frac{6 S_{D1} I}{1.4 R T_c^2}$ for $T_c > 4$ s I is importance facor; R is response reduction factor with values in the range of 2.5 to 3.0 for steel tanks.
AWWA D 100 *(Elevated tanks)*	**FEMA 368**
$(C_s)_i = \frac{1.25ZIS}{R_w T^{2/3}}$ $\ge \frac{0.75ZI}{R_w}$ $\le \frac{2.75ZI}{R_w}$ R_w is in the range of 2.0 to 3.0	$(C_s)_i = \frac{S_{DS} I}{1.4R}$ for $T < T_s$ $= \frac{S_{D1} I}{1.4RT}$ for $T_s < T_c \le 4$ s $= \frac{4 S_{D1} I}{1.4RT^2}$ for $T_c > 4$ s R is in the range of 2.0 to 3.0

Table 4: Expressions for base shear coefficients from AWWA D-110 and FEMA368

AWWA D110	FEMA 368
Impulsive mode $(C_s)_i = \dfrac{1.25ZIS}{R_i T_i^{2/3}} \le \dfrac{2.75ZI}{R_i}$	**Impulsive mode** $(C_s)_i = \dfrac{(0.6\frac{S_{Ds}}{T_0}T_i + 0.4S_{DS})}{1.4(R/I)}$ for $0 < T_i < T_0$ $= \dfrac{S_{DS}}{1.4(R/I)}$ for $T_0 \le T_i < T_s$ $= \dfrac{S_{D1}}{1.4(R/I)T_i}$ for $T_i \ge T_s$
Convective mode $(C_s)_c = \dfrac{4ZIS}{R_c T_c^{2}}$ R_i is response reduction factors for impulsive mode with values in the range of 2 to 2.75 and R_c is response reduction factor for convective mode; R_c = 1.0 for all types of tanks	**Convective mode** $(C_s)_c = \dfrac{6S_{D1}I}{T_c^{2}}$ Values of R are in the range of 1.5 to 3.0 for different types of concrete tanks

Table 5: Expressions for base shear coefficients from AWWA D-115 and FEMA368

AWWA D 115-95	FEMA 368
Impulsive mode $(C_s)_i = \dfrac{1.25ZIS}{R_w T_i^{2/3}}$ $\leq \dfrac{2.75ZI}{R_i}$	**Impulsive mode** $(C_s)_i = \dfrac{(0.6\frac{S_{Ds}}{T_0}T_i + 0.4S_{DS})}{1.4(R/I)}$ for $0 < T_i < T_0$ $= \dfrac{S_{DS}}{1.4(R/I)}$ for $T_0 \leq T_i < T_s$ $= \dfrac{S_{D1}}{1.4(R/I)T_i}$ for $T_i \geq T_s$
Convective mode $(C_s)_c = \dfrac{ZIS}{R_w T_c}$ R_w is response reduction factor with values in the range of 1.0-3.0.	**Convective mode** $(C_s)_c = \dfrac{6S_{D1}I}{T_c^{\,2}}$ Values of R are in the range of 1.5 to 3.0 for different types of concrete tanks

Table 6: Expressions for base shear coefficients from API 650 and FEMA 368

API 650	FEMA 368
Impulsive mode $C_i = 0.6\ Z\ I$	**Impulsive mode** $C_i = 0.24\ S_{DS}\ I$
Convective mode $C_c = \dfrac{0.75ZSI}{T_c}$ for $T_c \leq 4.5$ s $= \dfrac{3.375ZSI}{T_c^2}$ for $T_c > 4.5$ s Z is zone factor; I is importance factor; S is site coefficient	**Convective mode** $C_c = \dfrac{0.6S_{DS}I}{T_c}$ for $T_s < T_c \leq 4.0$ s $= \dfrac{2.396S_{DS}I}{T_c^2}$ for $T_c > 4.0$ s

Table 7: *Elastic spectrum* $S_e(T)$ and *Spectrum for linear analysis* $S_d\ (T)$ of Eurocode 8

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$S_e(T) = \alpha S\left[1+\frac{T}{T_B}(\eta\beta_0 - 1)\right]$

$0 \le T < T_B$

$= \alpha S\eta\beta_0$

$T_B \le T < T_C$

$= \alpha S\eta\beta_0\left[\frac{T_C}{T}\right]^{K1}$

$T_C \le T < T_D$

$= \alpha S\eta\beta_0\left[\frac{T_C}{T_D}\right]^{K1}\left[\frac{T_D}{T}\right]^{K2}$

$T_D \le T$

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$S_d\ (T) = \alpha S\left[1+\frac{T}{T_B}(\frac{\beta_0}{q} - 1)\right]$

$0 \le T < T_B$

$= \alpha S\frac{\beta_0}{q}$

$T_B \le T < T_C$

$= \alpha S\frac{\beta_0}{q}\left[\frac{T_C}{T}\right]^{Kd1}$

$(\ge 0.2\ \alpha)\ T_C \le T < T_D$

$= \alpha S\frac{\beta_0}{q}\left[\frac{T_C}{T_D}\right]^{Kd1}\left[\frac{T_D}{T}\right]^{Kd2}$

$(\ge\ 0.2\ \alpha)\ T_D \le T$

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where

$\alpha = a_g/g$; a_g=design ground acceleration ; β_0=Spectral acceleration amplification factor for 5% viscous damping; S = Soil parameter; η=Damping correction factor. η = 1.0 for 5% damping. For any other damping ξ, the value of $\eta = \{7/(2+\xi)\}^{0.5}$; K_1, K_2, K_{d1}, K_{d2} = Exponents which influence the shape of spectrum. Values of these exponents depend on soil condition. Their values for different soil conditions are given in Tables 4.1 and 4.2 of Eurocode 8 – Part 4; T_B, T_C = Limits of the constant spectral acceleration branch; T_D = Value defining beginning of the constant displacement range of the spectrum

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Table 8: Different types of tanks with their ductility factor, μ (Whittaker and Jury (2000))

Type of Tank	μ
Steel Tanks on Grade	
Elastically supported	1.25
Unanchored tank designed for uplift *(elephant foot shell buckling may occur under seismic overload)*	2.00[1]
Unanchored tank designed for uplift and elastic *(diamond shaped)* shell buckling mode	1.25
Anchored with non-ductile holding down bolts	1.25
Anchored with ductile tension yielding holding down bolts	3.00[2]
Ductile skirt pedestal	3.00[2]
On concrete base pad designed for rocking	2.00[2]
Concrete Tanks on Grade	
Reinforced Concrete	1.25
Prestressed Concrete	1.00
Tanks of other materials on Grade	
Timber	1.00
Non-ductile materials (eg. Fiberglass)	1.00
Ductile materials and failure mechanisms	3.00
Elevated Tanks	As appropriate for support structure [3]

Notes
1. Check that elastic buckling does not occur before elephant foot
2. Capacity design check required to protect against other forms of failure
3. Capacity design approach shall be used to protect elevated tanks against failure while yielding occurs in the chosen support system

Table 9: Correction factor, C_f (Whittaker and Jury (2000))

Ductility factor, μ	**Damping level, ξ (%)**						
	0.5	**1.0**	**2.0**	**5.0**	**10.0**	**15.0**	**20.0**
1.0	1.75	1.57	1.33	1.00	0.80	0.71	0.67
1.25	0.92	0.88	0.83	0.72	0.62	0.58	0.55
1.5	0.75	0.72	0.68	0.61	0.54	0.51	0.48
2.0	0.58	0.56	0.54	0.48	0.44	0.42	0.40
2.5	0.49	0.48	0.46	0.42	0.38	0.36	0.35
3.0	0.43	0.43	0.41	0.38	0.35	0.33	0.32
4.0	0.36	0.36	0.35	0.33	0.30	0.29	0.28

Table 10: Parameters for a low ductility tank

Code	Parameters
IBC 2000 and ACI 350.3	R = 1.5, I =1.25
Eurocode 8	q = 1.0, γ_I = 1.2
NZSEE guidelines	Sp = 1.0, μ = 1.25, ξ = 5%, C_f = 0.72

Table 11: Parameters for a high ductility tank

Code	Parameters
IBC 2000 and ACI 350.3	R = 3.0, I =1.25
Eurocode 8	q = 2.0, γ_I = 1.2
NZSEE guidelines	Sp = 1.0, μ = 3.0, ξ = 5%, C_f = 0.38

Table 12: Proposed values of Response reduction factor, R for IS 1893:2002

Tank	Proposed value of R
Ground supported tanks	
Unanchored steel tank	1.80
Anchored steel tank	2.25
Concrete tank with unconstrained flexible base	1.10
Concrete tank with non-sliding base	1.50
Elevated tanks	
Supported on RCC shaft	1.50
Supported on RCC frame staging	2.25
Supported on steel frame staging	2.25
Supported on masonry shaft	1.10

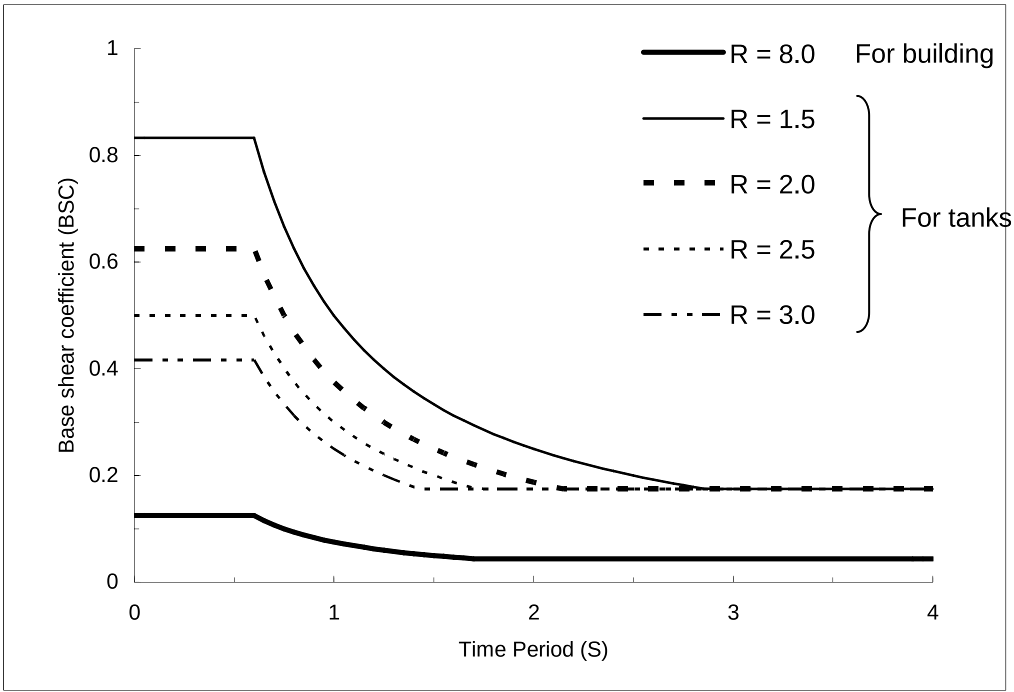


Figure 1: Variation of base shear coefficient with natural period; IBC 2000 (S_D =1.5, S_1 = 0.6, F_a =1.0, F_v = 1.5, Class D site)

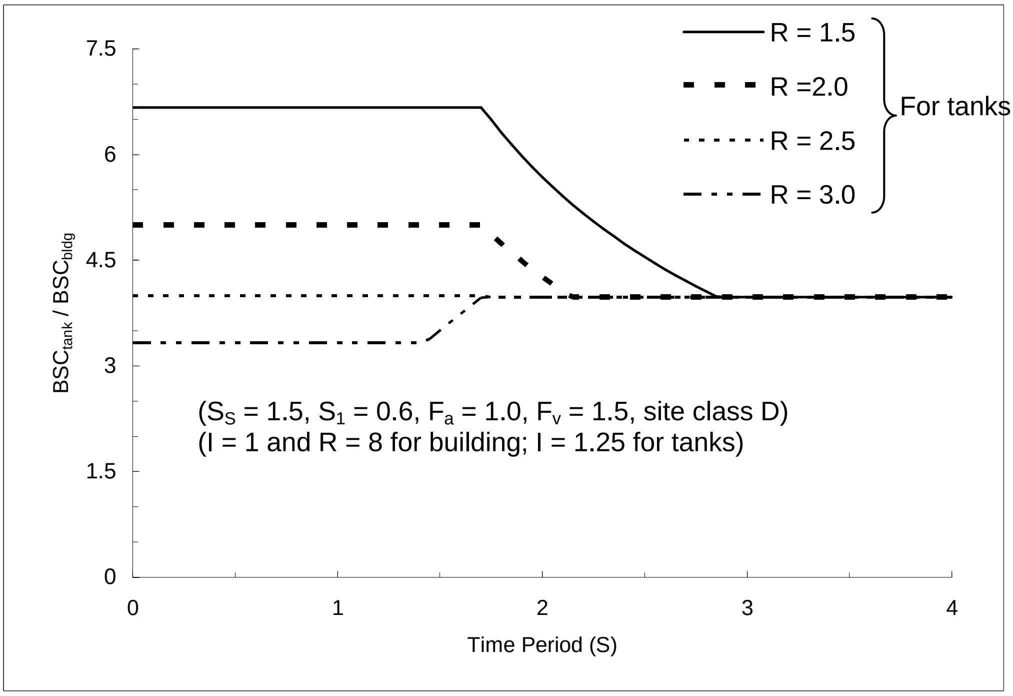


Figure 2: Ratio of tank and building base shear coefficient (IBC 2000)

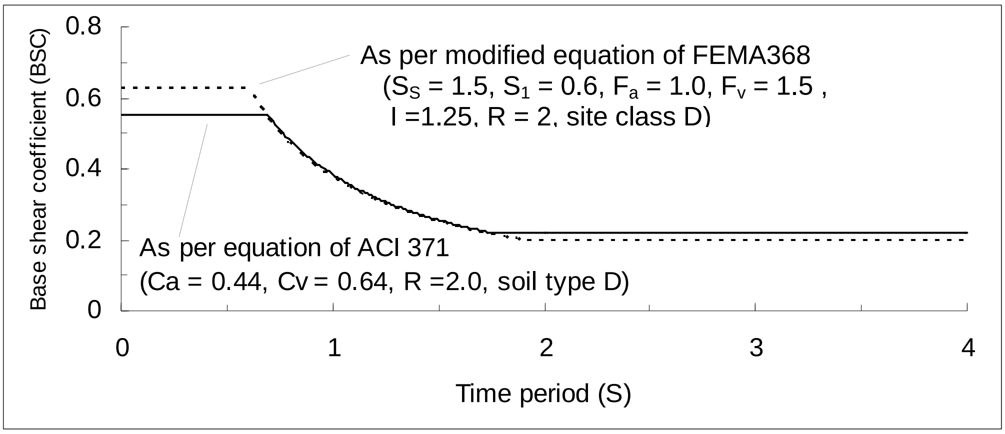


Figure 3: Comparison of base shear coefficient from ACI 371 and FEMA 368

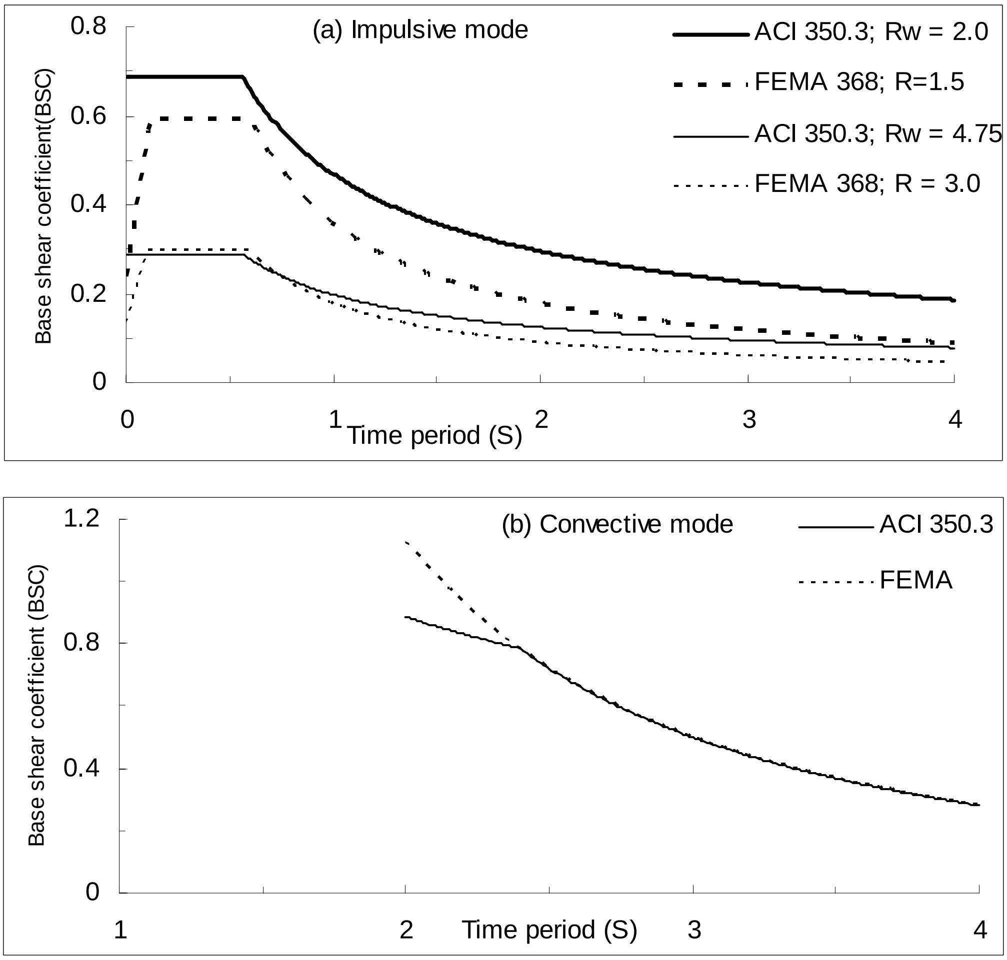


Figure 4: Comparison of base shear coefficient from ACI 350.3 and FEMA 368
(For ACI 350.3: Z = 0.4, S = 1.5, I = 1.25, soil type C;
For FEMA 368: S_S = 1.5, S_1 =0.6, F_a =1.0, F_v = 1.5, I = 1.25, site class D)

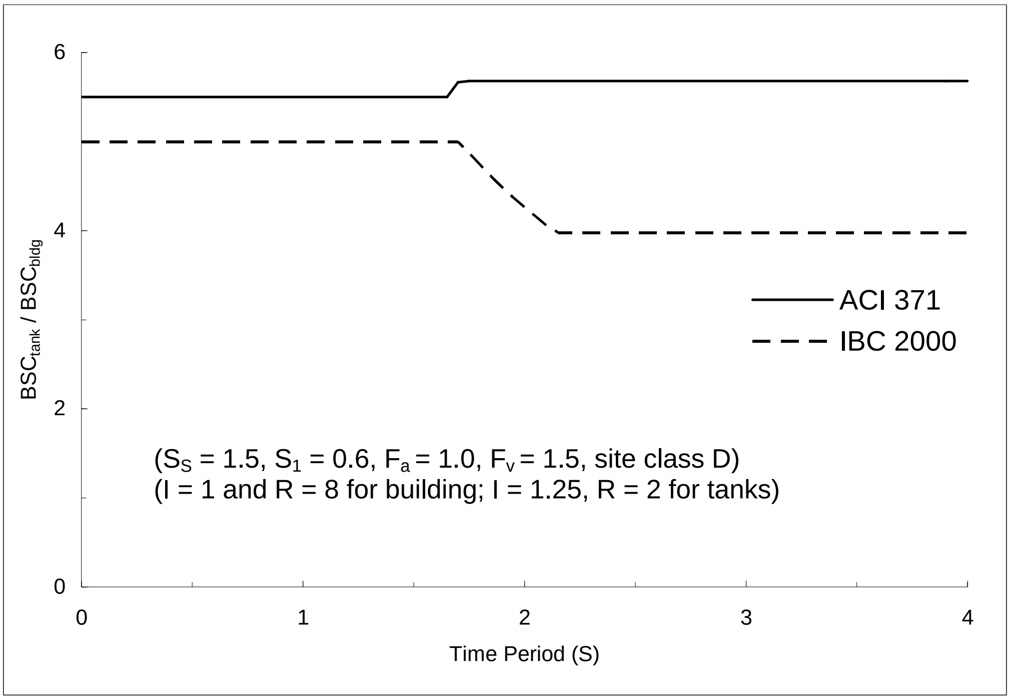


Figure 5: Ratio of tank and building base shear coefficient (ACI 371)

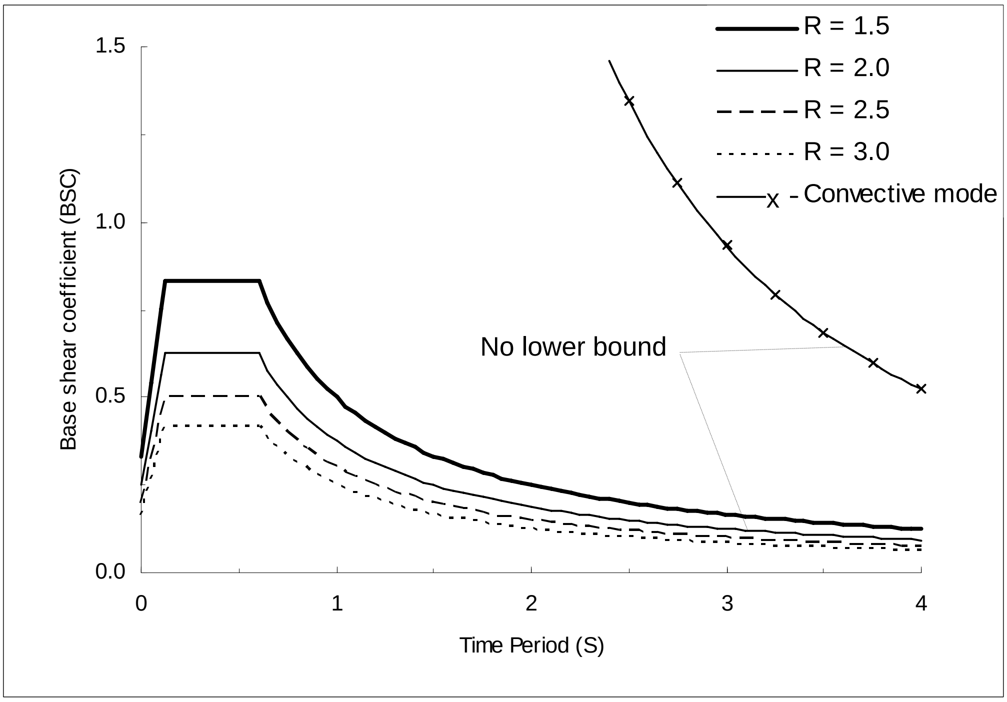


Figure 6: Variation of base shear coefficient with time period (ACI 350.3)
(S_D =1.5, S_1 = 0.6, F_a =1.0, F_v = 1.5, site class D)

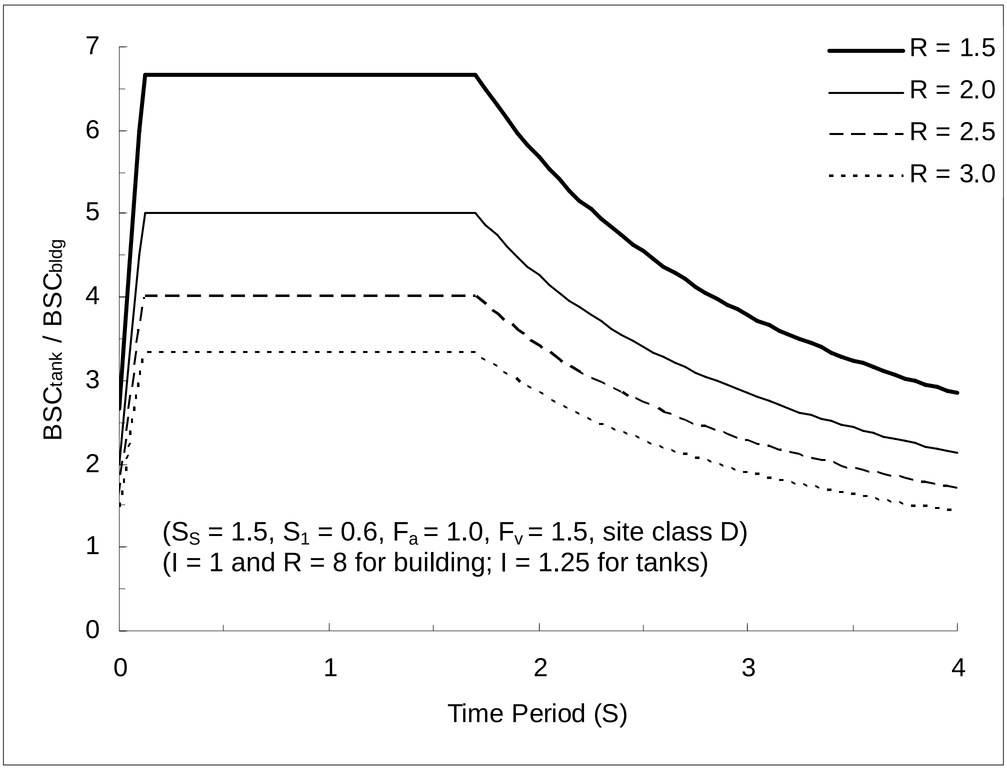


Figure 7: Ratio of base shear coefficient of tank (impulsive mode) and building (ACI 350.3)

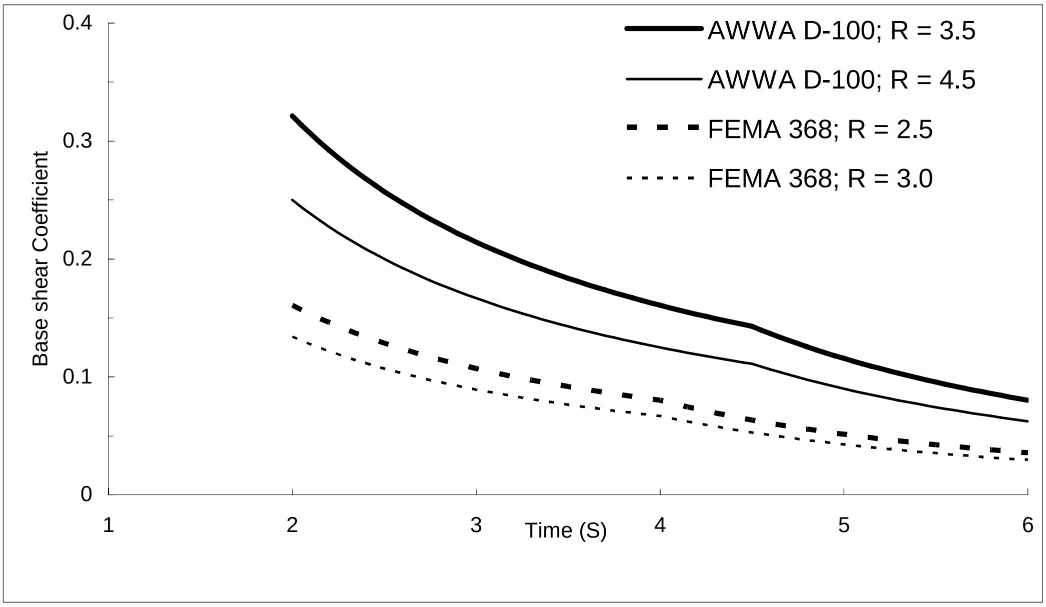


Figure 8: Comparison of convective base shear coefficient for ground-supported tank from AWWA D-100 and FEMA 368 (For AWWA D-100 Z = 0.4, S = 1.5, I = 1.25, soil type C; For FEMA 368: S_S = 1.5, S_1 =0.6, F_a =1.0, F_v = 1.5, I = 1.25, site class D)

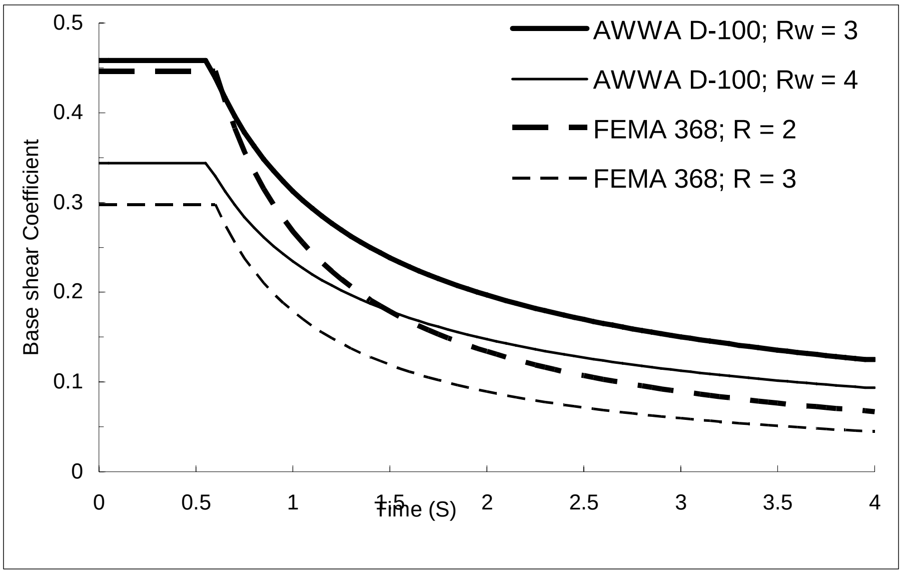


Figure 9: Comparison of impulsive base shear coefficient for elevated tank from AWWA D-100 and FEMA 368 (For AWWA D-100 Z = 0.4, S = 1.5, I = 1.25, soil type C; For FEMA 368: S_S = 1.5, S_1 =0.6, F_a =1.0, F_v = 1.5, I = 1.25, site class D)

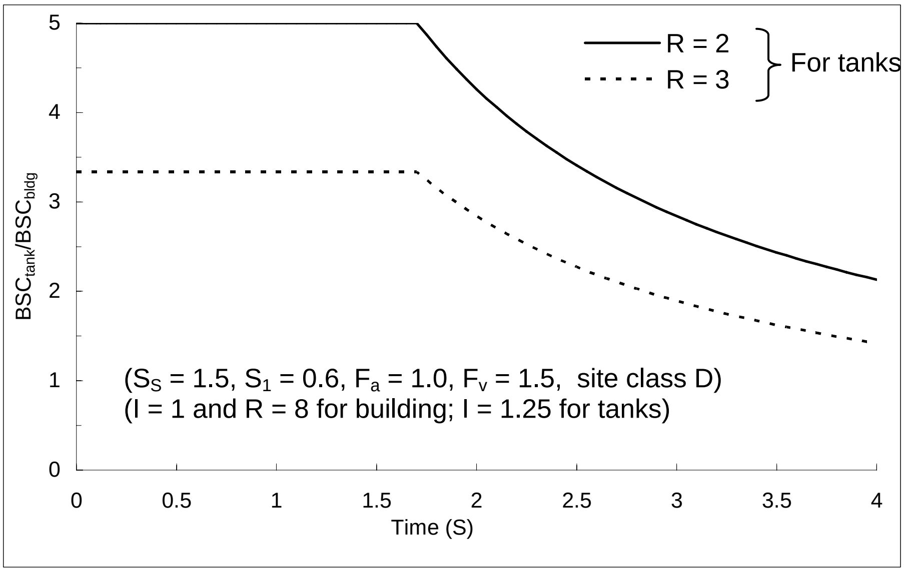
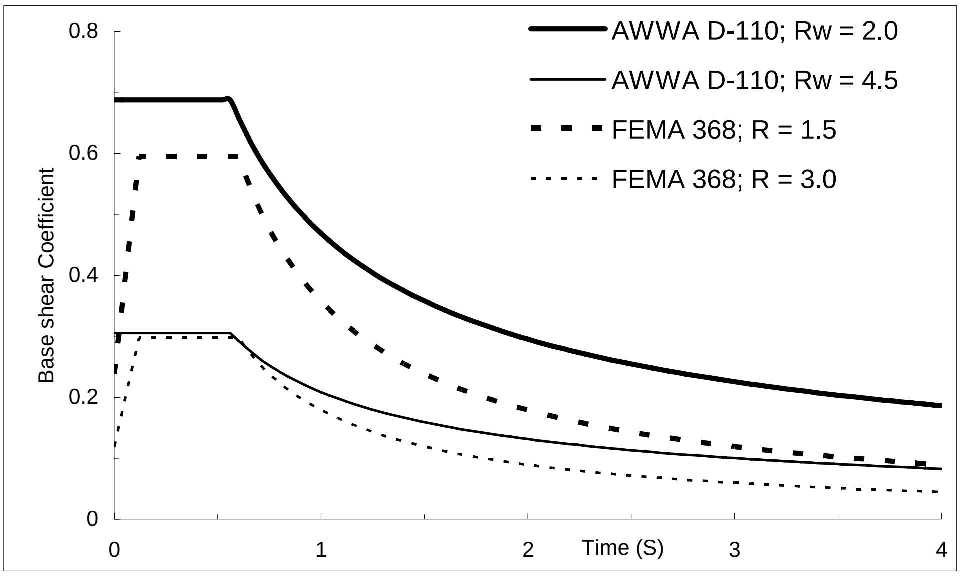
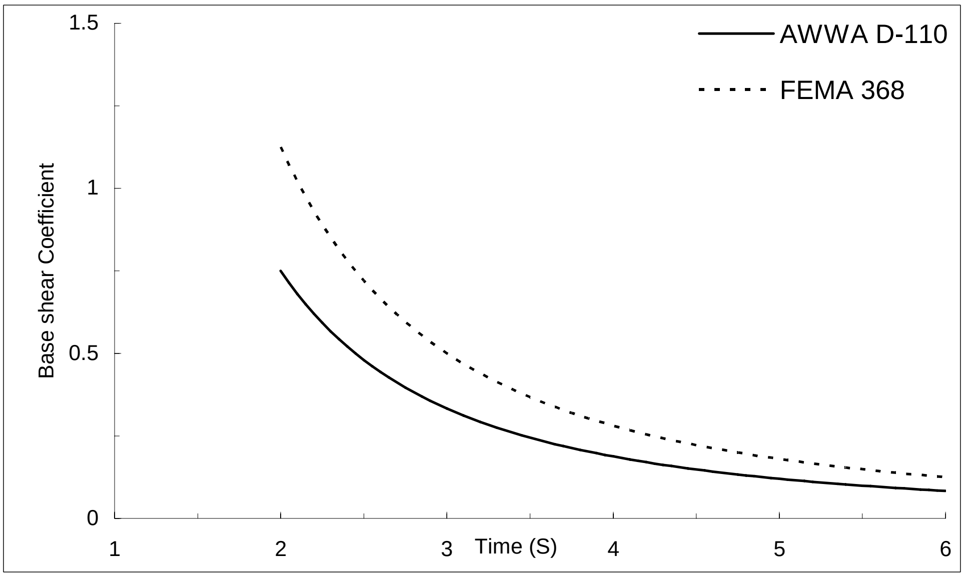


Figure 10: Ratio of elevated tank and building base shear coefficient (AWWA D-100)



(a) Impulsive base shear



(b) Convective base shear

Figure 11: Comparison of base shear coefficient from AWWA D-110 and FEMA 368 (For AWWA D-110 Z = 0.4, S = 1.5, I = 1.25, soil type C; For FEMA 368: S_S = 1.5, S_1 =0.6, F_a =1.0, F_v = 1.5, I = 1.25, site class D)

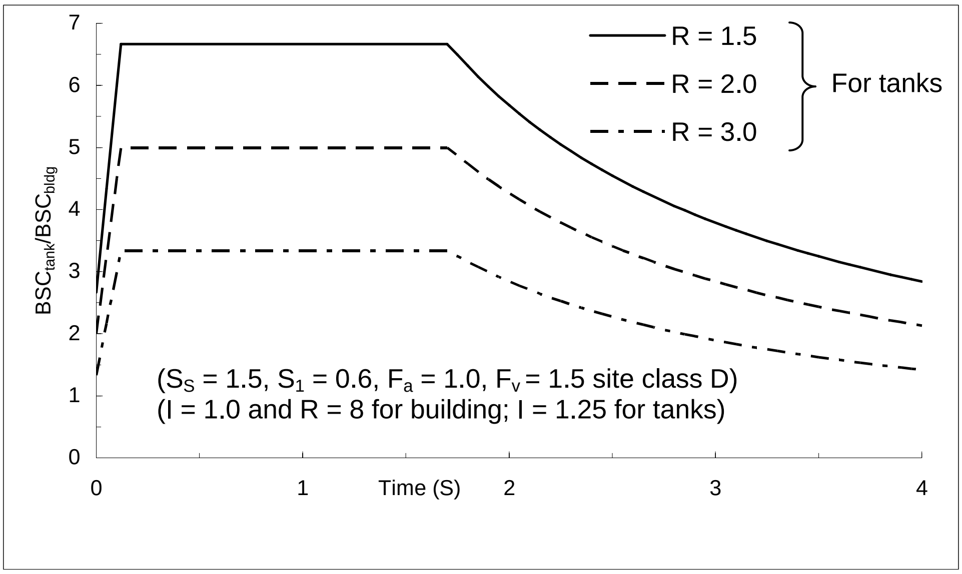


Figure 12: Ratio of tank and building base shear coefficient (AWWA D-110)

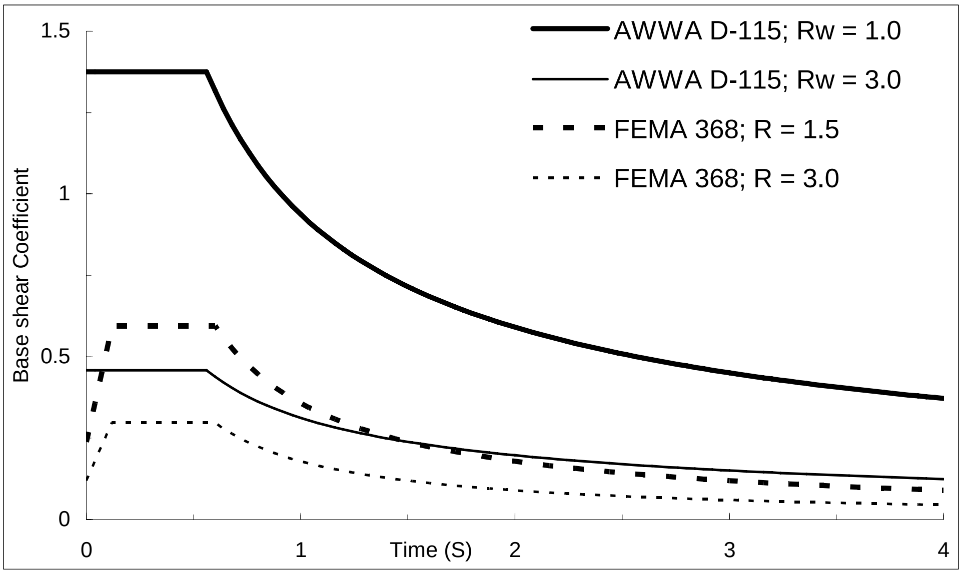


Figure 13: Comparison of impulsive base shear coefficient from AWWA D-115 and FEMA368

(For AWWA D-115: Z = 0.4, S = 1.5, I = 1.25, soil type C; For FEMA 368: S_S = 1.5, S_1 =0.6, F_a =1.0, F_v = 1.5, I = 1.25, site class D)

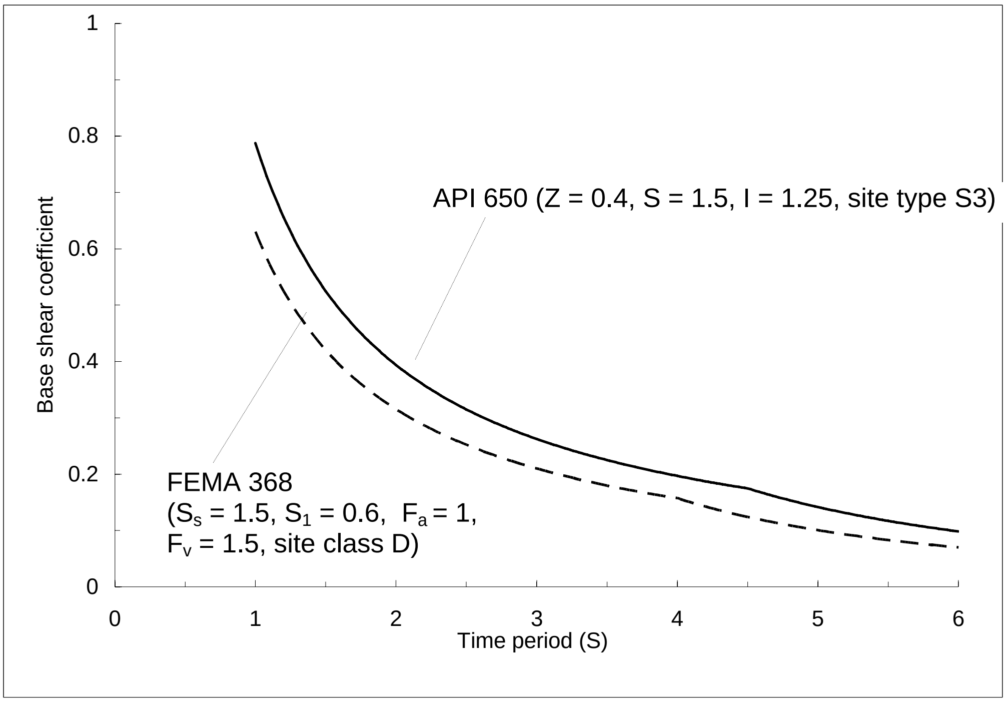


Figure 14: Convective base shear coefficient from API 650 and FEMA 368

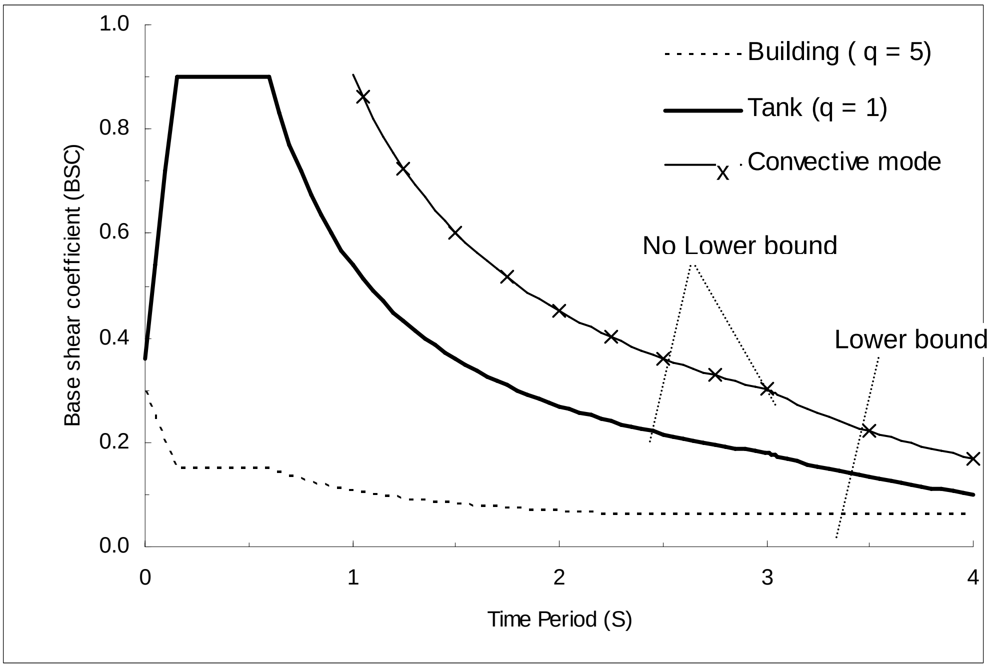


Figure 15: Variation of base shear coefficient with time period as per Eurocode 8 (S =1.0, β = 2.5, η= 1.0, K_1 =1.0, K_2 =2.0, K_{d1} =2/3, K_{d2} =5/3, T_B =0.1, T_C =0.4 T_D = 3.0, γ_I =1.0 for building and γ_I =1.3 for tank)

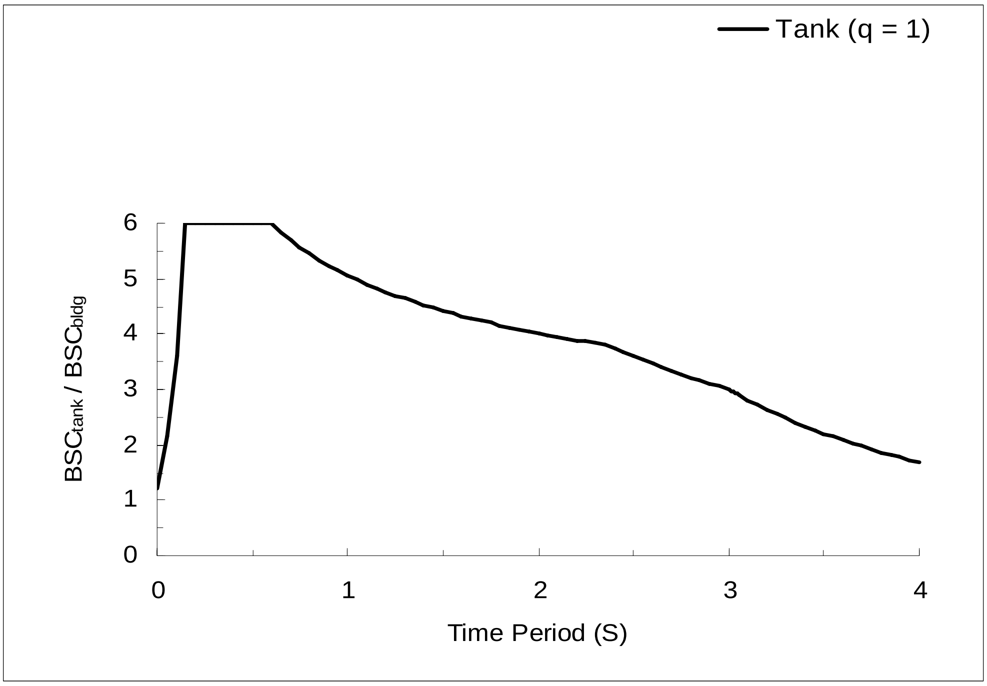


Figure 16: Ratio of base shear coefficient of tank impulsive mode and building (Eurocode 8)

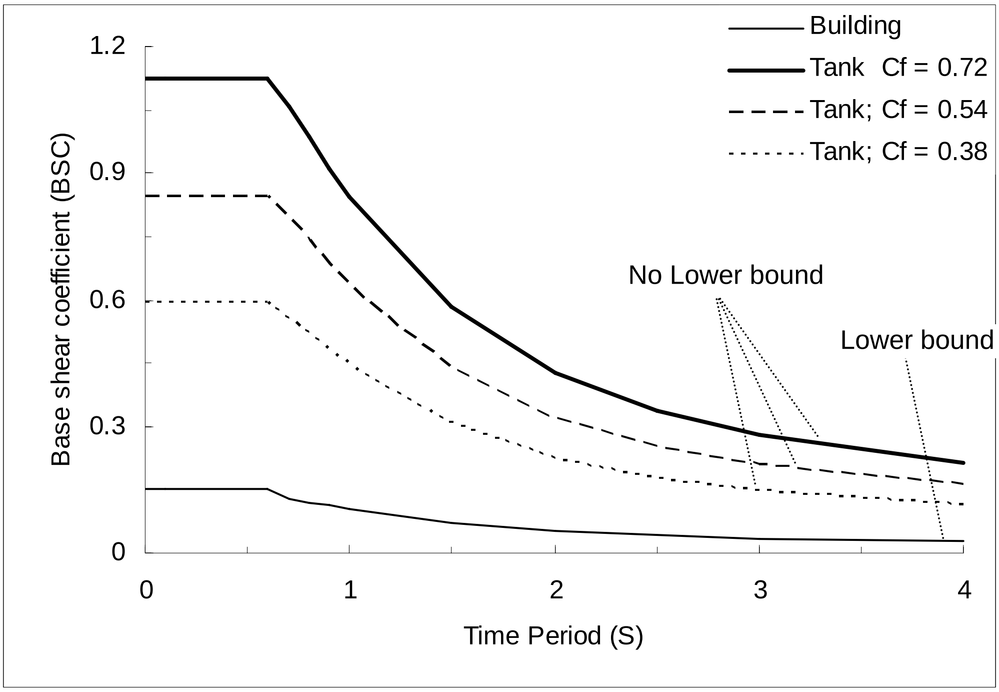


Figure 17: Variation of base shear coefficient with time period (NZSEE Guidelines) (S_p = 0.67 , R = 1.0 for building, S_p = 1.0 R =1.3 for tank, Z =1.2, L_u =1.0)

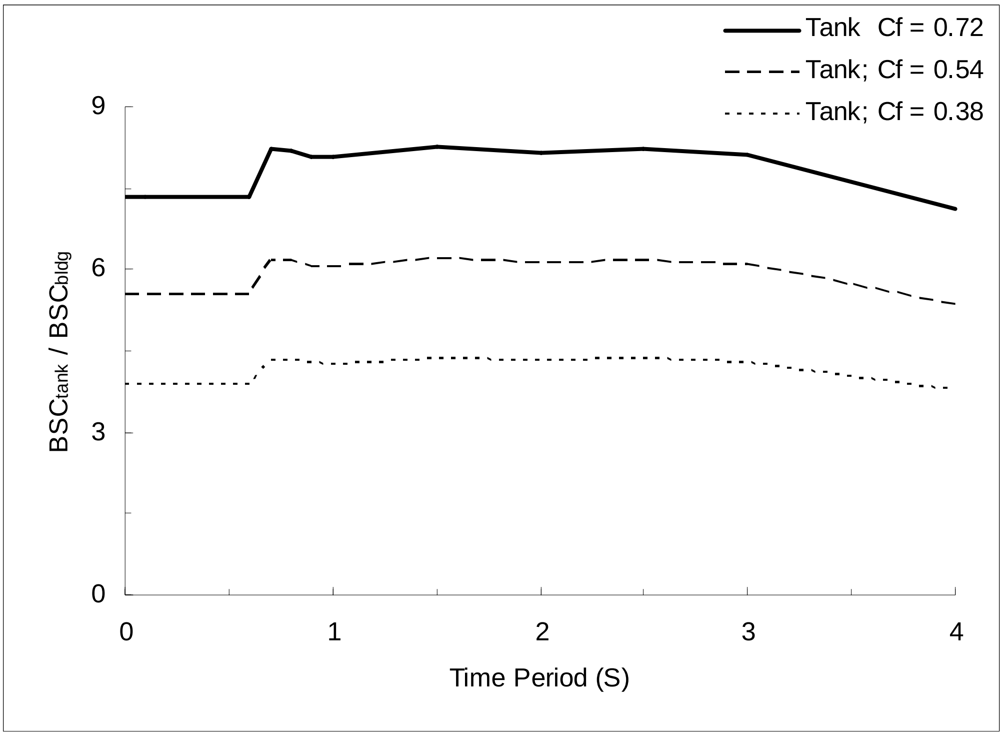


Figure 18: Ratio of base shear coefficient of tank and building (NZSEE Guidelines)

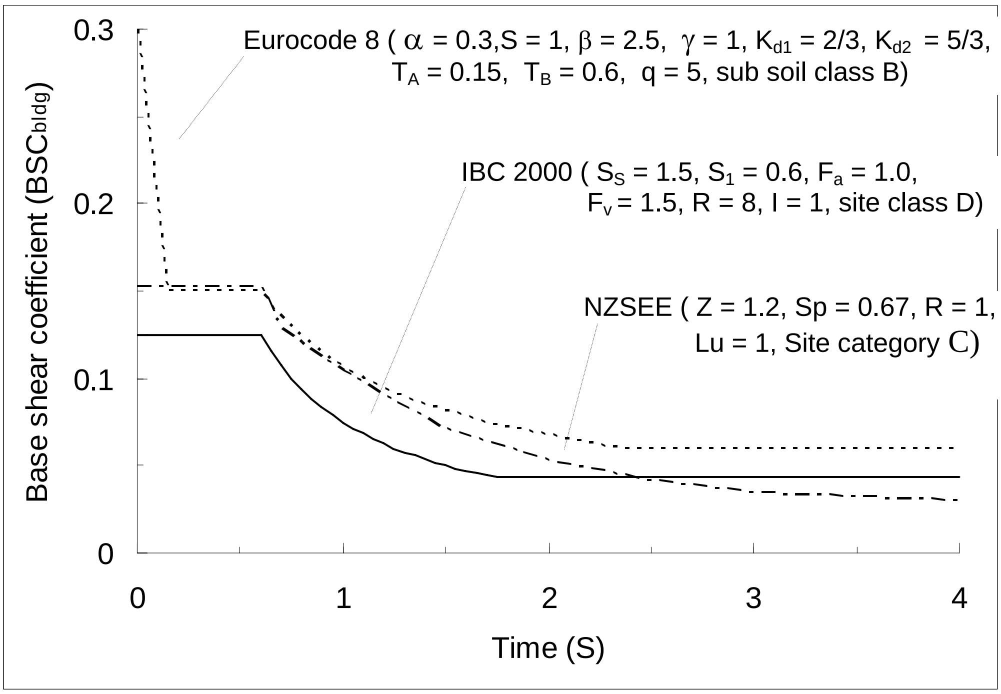


Figure 19: Comparison of base shear coefficient for ductile building obtained from various codes. Most severe zone in each code is considered

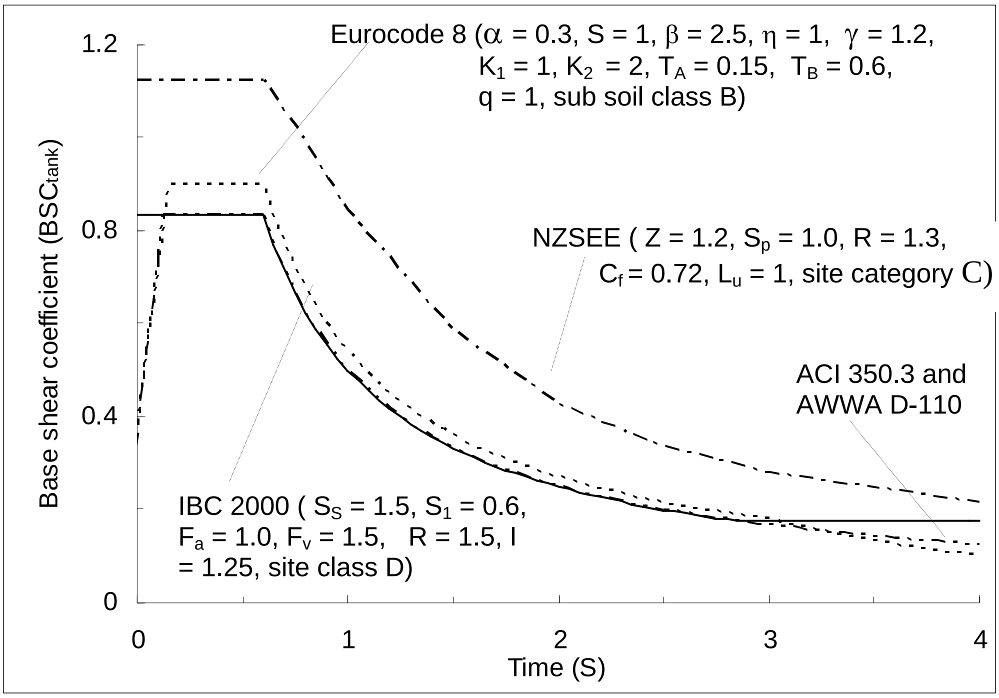


Figure 20: Comparison of base shear coefficient for ground supported unanchored concrete water tank obtained from various codes. Most severe zone in each code is considered.

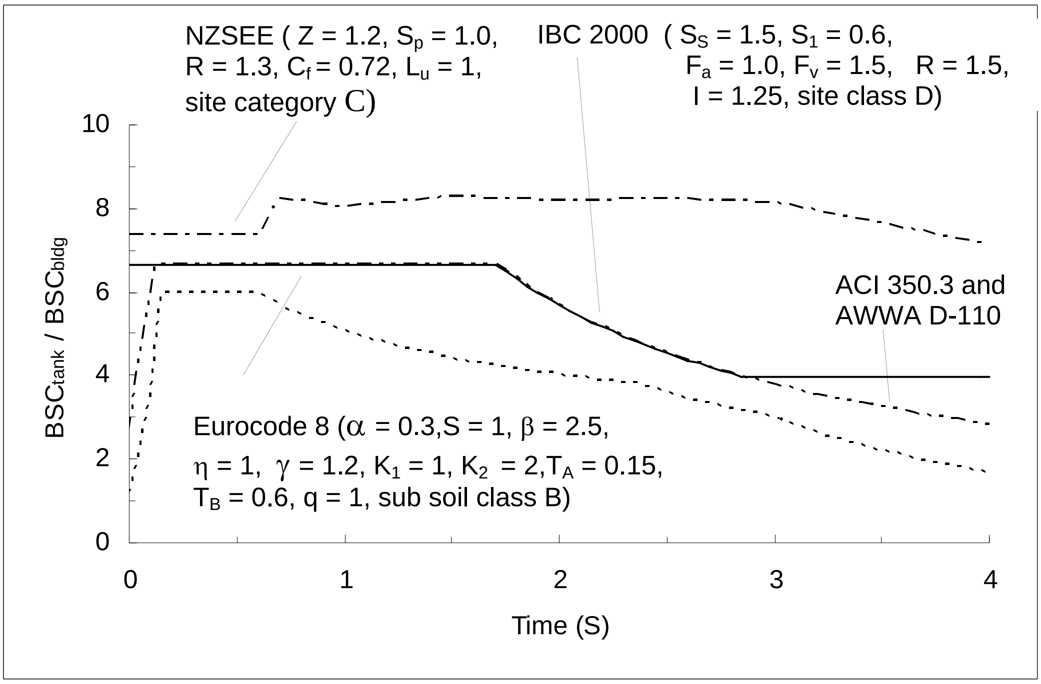


Figure 21: Comparison of ratio of base shear coefficients of tank and building from various codes (Low ductility tank).

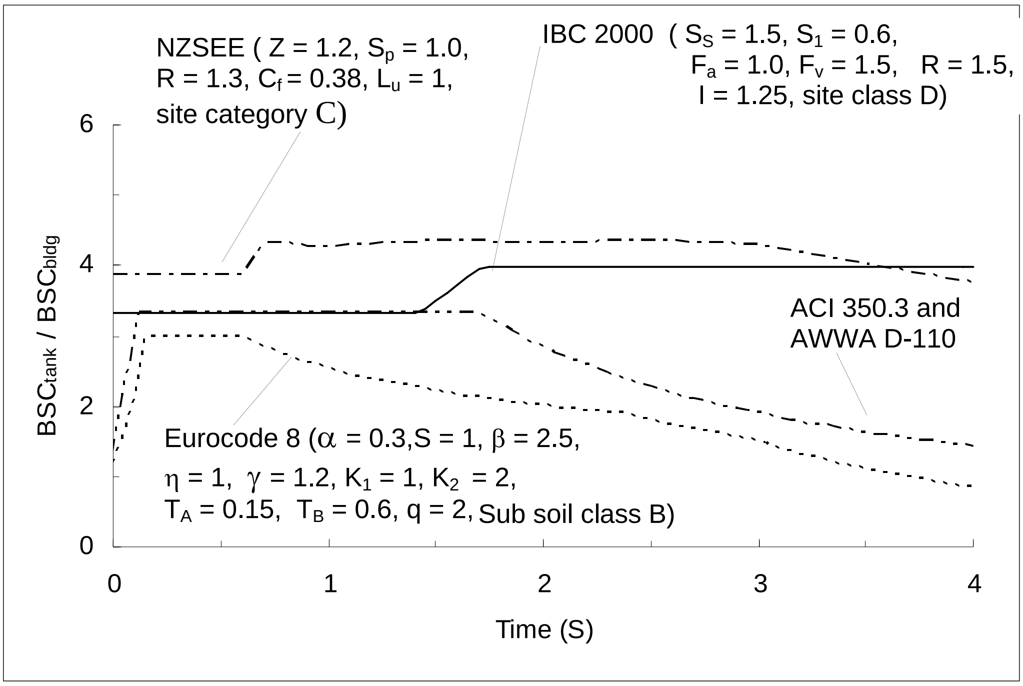


Figure 22: Comparison of ratio of base shear coefficient of tank and building from various codes (High ductility tank).

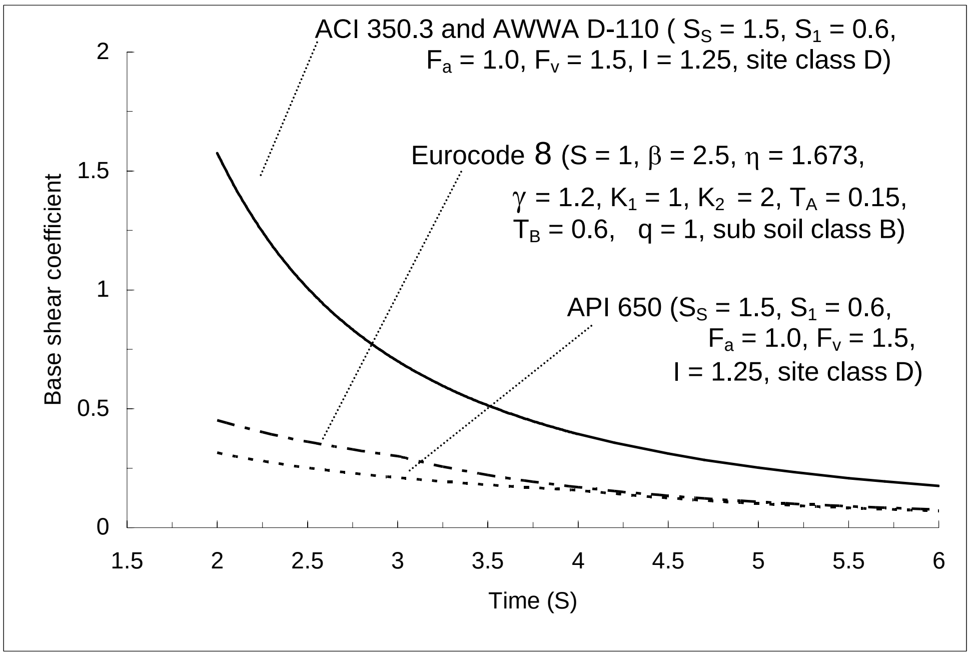


Figure 23: Comparison of base shear coefficient for convective mode

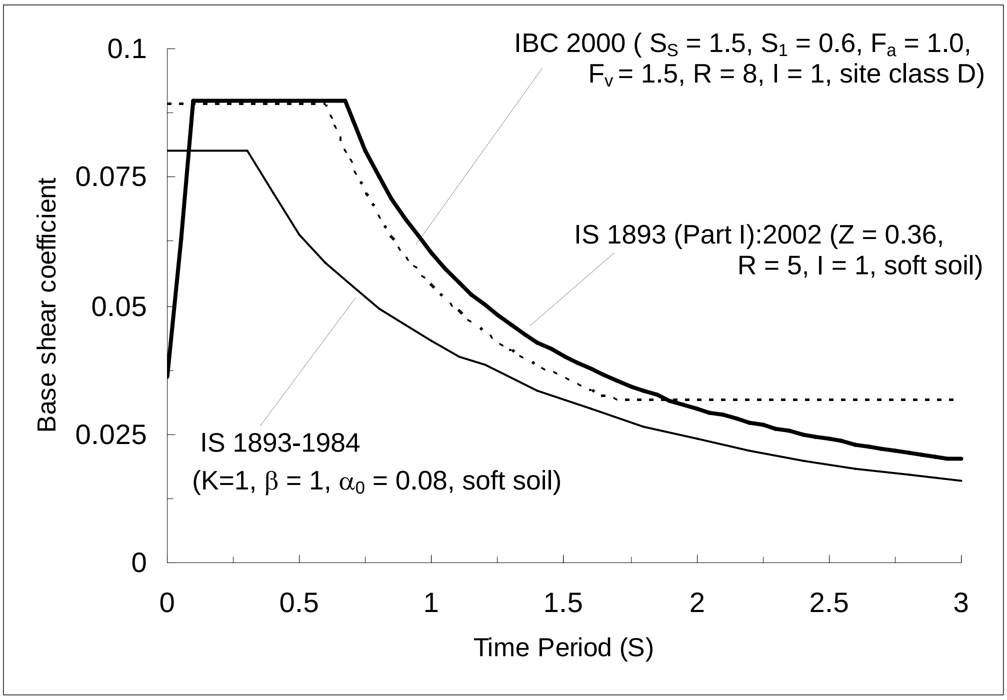


Figure 24: Comparison of base shear coefficient for building from IBC 2000, IS 1893:1984, IS 1893(Part 1):2002. IBC values are divided by 1.4 to bring them to working stress level.

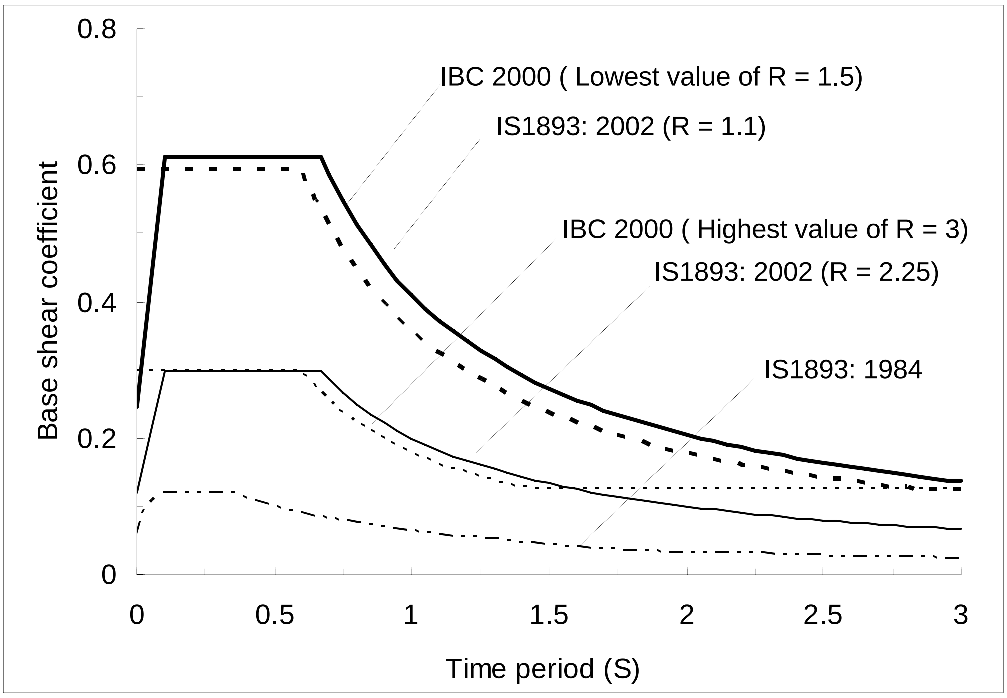


Figure 25: Base shear coefficients for tanks from IBC 2000, IS 1893:1984 and IS 1893(Part 1):2002. IBC values are divided by 1.4 to bring them to working stress level.