Masaaki Nakano

6 Renovation Design of Ageing Sewers as Composite Pipes by the Limit State Design Method

6.1 Application of the Limit State Design Method

Ageing in-service sewers requiring renewal tend to be in a structurally unsound condition because of concrete cracking, rebar corrosion and other anomalies arising from their long service history. Though these reinforced concrete structures were designed by the allowable stress method, which was the prevailing design theory at the time of construction, the critical stresses caused by cracks and other damages in existing sewers are likely to have already exceeded the allowable stress. For this reason, when renovating ageing sewers using the composite pipe method in which the existing sewer also serves as an important structural component, the allowable stress method may not be suitable for structural design because it focuses on the critical stress.

In contrast, the limit state design method focuses on the limit states of damage or failure that a structure may experience during its design life, and the criterion for a safe design is to ensure that such a limit state is not reached. For reinforced concrete structures, the limit states can be classified into:

1. The ultimate limit state: This involves the collapse of the whole structure or its elements under the design loads.
2. Serviceability limit states: The structure is unfit to serve its normal function due to excessive deformation, cracking or vibration.
3. Special limit states: This type of limit state mainly concerns damage or failure due to abnormal conditions such as strong earthquake, structural effects of corrosion or deterioration, and structural effects of other extreme conditions. Obviously, the relevance of a special limit state to structural design must be justifiable.

As such, limit state design is a performance-verification design method. In the design process, partial safety factors are employed with respect to load, material, structural analysis, structural member, and structural importance, and the influence of these factors on the required performance concerning a limit state can be evaluated separately. Therefore, important design considerations can be clearly identified so that a comprehensive evaluation can be carried out with respect to multiple performance requirements. Table 6.1 shows typical performance requirements, limit states and check items. In the case of sewer renovation, structural safety can be verified...
by evaluating the possibility of cross-sectional failure of the renovated structure under design loads, and serviceability can be checked by evaluating its functional sustainability and trafficability. Seismic safety can be verified by performing dynamic structural analysis under Level 1 and Level 2 earthquake loading.

Table 6.1: Examples of performance requirements, limit states and check items

<table>
<thead>
<tr>
<th>Performance requirements</th>
<th>Limit states</th>
<th>Check item</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safety</td>
<td>Cross-sectional failure</td>
<td>Cross-sectional force</td>
</tr>
<tr>
<td></td>
<td>Fatigue failure</td>
<td>Stress, cross-sectional force</td>
</tr>
<tr>
<td></td>
<td>Structural instability</td>
<td>Deformation, deformation due to foundation structure</td>
</tr>
<tr>
<td>Serviceability</td>
<td>Appearance</td>
<td>Crack width, stress</td>
</tr>
<tr>
<td></td>
<td>Noise, vibration</td>
<td>Noise/vibration level</td>
</tr>
<tr>
<td></td>
<td>Trafficability, etc.</td>
<td>Displacement, deformation</td>
</tr>
<tr>
<td></td>
<td>Watertightness</td>
<td>Permeability of structure, crack width</td>
</tr>
<tr>
<td></td>
<td>Functional damage</td>
<td>Force, deformation, etc.</td>
</tr>
<tr>
<td>Restorability</td>
<td>Repairability</td>
<td>Force, deformation, etc.</td>
</tr>
</tbody>
</table>

6.2 Basic Concept of Performance Verification

Performance verification of renovated sewer pipes by the limit state design method involves two types of loading condition, normal loading and earthquake loading. This section outlines the basic concept of limit state design, using the sectional forces of a renovated sewer (moment, shear, axial force) as design indices.

The performance of the structure of interest in the ultimate limit state is checked by comparing the response value under the design load with the capacity value under the failure load. Specifically, the basic criterion is that the sectional force under the design load multiplied by a structure factor must not exceed the sectional capacity determined from the ultimate failure analysis of the structure, which is expressed as

\[
\gamma_i \frac{S_d}{R_d} \leq 1.0
\]

(6.1)

where \(\gamma_i\): structure factor which measures the degree of importance of the structure; \(S_d\): design sectional force; and \(R_d\): design sectional capacity.

The design sectional force \(S_d\) is expressed as
Performance Requirements for Renovated Sewer

6.3 Performance Requirements for Renovated Sewer

6.3.1 Under normal loading

(1) Serviceability limit state
Sewer pipes are required to maintain watertightness under normal loading, and sewer renovation must not accelerate the deterioration of the existing pipe. New cracking in the renovated sewer must therefore be prevented.

(2) Ultimate limit state
Sewer pipes are required to maintain their discharge function even under rare loading conditions encountered during their service life. Therefore, the renovated sewer must not fail even under a large load exceeding the design load and must maintain sufficient load-carrying capacity as a structure.
Renovation Design of Ageing Sewers as Composite Pipes by the Limit State Design Method

Figure 6.1: Flow of performance verification of composite pipe based on limit state design method
(3) Fatigue limit state
Because sewer pipes are underground structures, live loads do not act directly on them, so it is highly unlikely that sewer pipes will be endangered by fatigue failure caused by cyclic loading. Accordingly, fatigue strength is not taken into consideration as a performance requirement for renovation design.

6.3.2 Under earthquake loading

Earthquake resistance requirements for existing and renovated sewers are as specified in the Japan Sewage Works Association’s *Guidelines for Seismic Design and Retrofit of Sewerage Facilities* (hereafter referred to as the JSWA Seismic Guidelines) (JSWA 2014). As shown in Table 6.2, performance requirements for sewerage pipelines under earthquake loading are broadly classified according to earthquake ground motion levels.

<table>
<thead>
<tr>
<th>Pipeline</th>
<th>Design ground motion</th>
<th>Seismic performance required</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level 1</td>
<td>Level 2</td>
</tr>
<tr>
<td>Existing重要干管, etc.</td>
<td>R</td>
<td>R</td>
</tr>
<tr>
<td>Other pipeline</td>
<td>R</td>
<td>NR</td>
</tr>
<tr>
<td>New Important干管, etc.</td>
<td>R</td>
<td>R</td>
</tr>
<tr>
<td>Other pipeline</td>
<td>R</td>
<td>NR</td>
</tr>
</tbody>
</table>

Note: R = required; NR = not required

Level 1 earthquake ground motion refers to earthquake ground motion of a magnitude that is likely to occur several times during the service life of a structure. Level 2 earthquake ground motion refers to very strong earthquake ground motion, and its probability of occurrence is very small during the service life of a structure.

(1) Under Level 1 earthquake ground motion: serviceability limit state
Sewer pipes are required to maintain their design discharge capacity even after earthquake ground motion (Level 1 earthquake ground motion) that is likely to be encountered several times during the service life of the pipelines. Therefore, performance verification should ensure that no large structural deformation of renovated sewers occurs under such earthquake conditions.
(2) Under Level 2 earthquake ground motion: ultimate limit state
Sewer pipes are required to maintain their discharge function even after very strong earthquake ground motion (Level 2 earthquake ground motion) that can occur during their service life, although its probability of occurrence is low. Therefore, performance verification should ensure that no structural failure of renovated sewers occurs under such earthquake conditions.

6.4 Performance Verification under Normal Loading

This section describes in detail the methods for verifying the safety of renovated sewers under normal loading. Nonlinear structural analysis of composite pipe is performed to verify structural performances at the serviceability limit state and the ultimate limit state, using the no-tension interface modelling approach introduced in Chapter 5.

The limit states defined under normal loading and their verification criteria are as follows (JSWA, 2011):

For serviceability limit state: New cracking must not occur anywhere in the renovated sewerage structure under the design load.

For ultimate limit state: The critical sectional force of the renovated sewerage structure under the design load must not exceed the sectional capacity or ultimate strength of the structure.

6.4.1 Verification for serviceability limit state

With respect to the serviceability limit state of a renovated sewer, the design sectional moment under the sustained load and traffic load must not exceed the cracking moment of the cross section. Specifically, the maximum tensile stress occurring in the renovated structure must not exceed the tensile strength of the material.

6.4.2 Verification for ultimate limit state

Sectional forces which are taken into consideration regarding the ultimate limit state of a renovated sewer are bending moment and shear force. This means that the design sectional force $S_d$ and the design sectional capacity $R_d$ expressed by Eqs. (6.2) and (6.3), respectively, can be rewritten as:

\[
S_d : \begin{cases} 
M_d = \gamma_a \gamma_f M \\
V_d = \gamma_a \gamma_f V
\end{cases} \quad R_d : \begin{cases} 
M_{rd} = M_u / \gamma_m / \gamma_b \\
V_{rd} = V_u / \gamma_m / \gamma_b
\end{cases}
\] (6.4)
where \( M, M_d \): bending moment under the design load, design bending moment; \( V, V_d \): shear force under the design load, design shear force; \( M_u, M_{rd} \): maximum bending moment at failure, design moment capacity; and \( V_u, V_{rd} \): maximum shear at failure, design shear capacity.

### 6.4.3 Safety factors

Table 6.3 shows typical safety factor values indicated in the JSCE *Standard Specifications for Concrete Structures: Design* (hereafter referred to as the JSCE *Standard Specifications*) (JSCE, 2012):

<table>
<thead>
<tr>
<th>Safety factors</th>
<th>Material factor ( \gamma_m )</th>
<th>Member factor ( \gamma_b )</th>
<th>Structural analysis factor ( \gamma_a )</th>
<th>Load factor ( \gamma_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete ( \gamma_c )</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Steel ( \gamma_s )</td>
<td>1.0 or 1.051.1–1.3</td>
<td>1.0</td>
<td>1.0–1.2</td>
<td>1.0–1.2</td>
</tr>
</tbody>
</table>

(1) Material factor \( \gamma_m \):
The material factor is to be determined by taking into consideration factors such as a change in an undesirable direction from the characteristic value of material strength, differences in material properties between test specimens and structure materials, the influence of material properties on the limit state and changes over time in material properties. Such an evaluation needs to be made for the materials used in each renovation method.

(2) Member factor \( \gamma_b \):
The member factor is to be determined by taking into consideration factors such as the uncertainty of the member strength calculation, the effect of variability of member dimensions, and the degree of importance of members, that is, the influence that the member of interest has when it reaches a limit state on the entire structure. The evaluation needs to take into consideration the fact that a composite pipe is a renovated structure consisting partially of existing sewer members that are not in a sound condition.

(3) Structural analysis factor \( \gamma_a \):
The structural analysis factor is to be determined by taking into consideration the uncertainty of the calculation of the design sectional force and the design sectional
capacity through nonlinear analysis. The evaluation needs to take into consideration the accuracy of analytical results relative to past test results.

(4) Load/action factor $\gamma_f$:
The load factor is to be determined by taking into consideration factors such as a change in an undesirable direction from the characteristic value of load (action), the uncertainty of the load calculation, changes in load during the design service life, and the influence of load characteristics on the limit state. Unlike in the case of new construction, the primary load acting on the renovated sewer is a load acting through soil compacted to a certain degree. Accordingly, the evaluation needs to take into consideration the uncertainty of fact finding and the variability of obtained data.

(5) Structure factor $\gamma_i$:
The structure factor is to be determined by taking into consideration factors such as the degree of importance of the structure, the socio-economic impact anticipated when the limit state is reached, and economic factors including reconstruction or repair cost. Since there is no established method for renovating these composite pipes in future, the evaluation needs to take into consideration the necessity of reliably rehabilitating sewer pipes.

### 6.4.4 Loads to be considered

Table 6.4 lists the loads to be taken into consideration at the design stage.

**Table 6.4: Loads to take into consideration when designing composite pipes**

| Dead load                  | Weight of members       | R  |
|                           | Weight of water in pipe | CR |
| Live load                 | Live load from above    | R  |
|                           | Impact                  | R  |
| Earth pressure            | Vertical earth pressure | R  |
|                           | Horizontal earth pressure | R  |
|                           | Earth pressure due to live load | R  |
| Groundwater pressure      |                         | CR |
| Buoyancy                  |                         | CR |

Note: R (required): load that must be taken into consideration; CR (conditionally required): load that does not need to be taken into consideration except in cases where its influence is expected to be significant.
6.4.5 Structural analysis model

As discussed in Chapter 5, the cross section of a renovated sewer pipe is modelled by using two-dimensional plane elements. As for materials, the nonlinearity of concrete, steel and lining materials is taken into account. By using the nonlinear finite element analysis based on a fracture mechanics model, the behaviour of the pipe from the occurrence of cracking to the ultimate failure is analysed.

6.4.6 Nonlinear structural analysis

The design of a composite pipe is characterised by calculation of the ultimate limit state using nonlinear structural analysis for accurately evaluating the load-carrying capacity of the pipe. Figure 6.2 shows the differences in the safety verification methods with respect to sectional or structural failure applied to a typical reinforced concrete structure and a composite pipe. As seen, the fundamental difference between the two is the method of capacity calculation. While for the design of reinforced concrete structures, member capacity can be determined from theoretical equations based on idealised structural mechanics models, in the renovation design of ageing sewers it is obtained through numerical analysis.

The principal loads which act on the structure are classified as described below according to overburden thickness:

(a) For overburden thickness of less than 4 m
Assume that primary loads are live loads (traffic loads) and increase the primary loads by increasing the load weighting factor of the live loads till structural failure. In the following, the load weighting factor is referred to as the load coefficient. The load coefficient obtained at failure shows the ratio of the failure load to the design load.
- Load step 1: Apply dead loads, which include earth and groundwater pressures and self-weight.
- Load step 2: Apply live loads incrementally until the design load is reached.
- Load step 3: While keeping dead loads constant, apply live loads by gradually increasing the load coefficient until structural failure is reached.

(b) For overburden thickness of 4 m or greater
Assume that primary loads are the earth and groundwater pressures and the live loads, and increase the primary loads by increasing the load coefficient till structural failure. The load coefficient obtained at failure shows the ratio of the failure load to the design load.
- Load step 1: Apply self-weight.
- Load step 2: Apply primary loads incrementally until the design load is reached.
Load step 3: While keeping the self-weight of the pipe constant, apply the primary loads by gradually increasing the load coefficient until structural failure is reached.

**Figure 6.2:** Comparison of computational methods for ultimate limit state between reinforced concrete structures and composite pipes

In the verification with respect to the serviceability limit state, checks are made, by following the steps described above, to determine whether cracking occurs under the design load. In the verification with respect to the ultimate limit state, the design sectional capacity and the design sectional force are calculated as follows:

1. The maximum sectional force (design sectional capacity) of each member (such as the top plate, the bottom plate, or the sidewalls) at structural failure is calculated, and its location in that member is recorded.
2. The sectional force (design sectional force) at the abovementioned location under the design load is calculated.
Figure 6.3 shows the analysis procedure for evaluating the ultimate load-carrying capacity by incremental loading. It should be pointed out that the maximum sectional force obtained as described above may not necessarily be the actual capacity of that member, because under the designated primary loads the critical failure mode of that member may not occur.

**Figure 6.3:** Flow of performance verification for renovation design of ageing sewers (example: overburden thickness less than 4 m)
6.4.7 Performance evaluation in terms of load coefficients

(1) Basic concept
Conventional practice when designing a reinforced concrete pipe for sewers is to determine the maximum bending moment occurring in the pipe under the design load based on the type of foundation and to select a pipe type so that the moment remains on the safe side relative to the allowable bending moment for the pipe (JSWA, 2003). The moment capacity for a pipe can be calculated by adding the bending moment due to the self-weight of the pipe to the maximum bending moment occurring under the cracking load determined according to the code-specified values. Thus, the design approach uses the margin of safety of the cracking load relative to the design load as a safety factor, which is comparable to the load coefficient introduced above. Therefore, performance evaluation based on the margin of safety relative to the design load is a suitable method for evaluating the structural safety of sewer pipes.

In the design of composite pipes, the design load is increased incrementally through the load coefficient, and structural strength is evaluated by nonlinear structural analysis. Consequently, the margin of safety relative to the design load with respect to the cracking load and the failure load can be evaluated directly. Therefore, besides the verification method based on sectional forces, the verification method based on the load coefficient has also been in use.

The JSWA Design and Construction Guidelines for Sewer Pipe Rehabilitation (hereafter referred to as the JSWA Guidelines) shows examples of load-carrying capacity evaluation using the load coefficient as a safety factor when designing composite pipes by the limit state design method.

To illustrate this load coefficient design concept, consider the design of a member subjected to axial force by the limit state design method. The ultimate limit state is verified by

\[ \gamma_i \gamma_a \gamma_f F_k \leq 1.0 \]

which leads to

\[ \frac{Af_k}{F_k} \geq \gamma_i \gamma_a \gamma_b \gamma_m \gamma_f \quad \text{or} \quad \frac{F_u}{F_k} \geq \gamma_i \gamma_a \gamma_b \gamma_m \gamma_f \]

(6.6)

where \( F_k \): the characteristic value of load; \( f_k \): the characteristic value of material strength; \( A \): the effective cross-sectional area of the member; and \( F_u \): the ultimate sectional capacity or failure load.

Clearly, Eqs. (6.5) and (6.6) are equivalent so the verification of the ultimate limit state can be performed by checking the ratio of the failure load to the characteristic value of the load. It can be shown that for linear elastic materials, this statement is also true for the design of members under bending and shear loads. If nonlinearity in the stress-strain relation is weak, it then becomes approximately true.

Figure 6.4 shows the simple relationship between the sectional force-based verification method and the load coefficient-based verification method. Using the
load coefficient, the performance requirements for renovation design of ageing sewers can be expressed as

For serviceability limit state: $\lambda_c > 1.0$
For ultimate limit state: $\lambda_u \geq 2.5$

Note that $\lambda_c$ is the load coefficient for cracking, and $\lambda_u$ is the ultimate load coefficient at structural failure.

**Nonlinear structural analysis**

- Calculation of design sectional force, $S_d$
- Calculation of design section capacity, $R_d$, and failure load, $F_u$

### Characteristic value of material strength

$f_k = f_c / \gamma_m$

### Design strength of material

$f_d = f_k / \gamma_m$

### Sectional capacity

$R(f_d) = R(f_k / \gamma_m) = R(f_k) / \gamma_m$

### Design sectional capacity

$R_d = R(f_k) / \gamma_h$

Equivalent to $R(f_k) / (\gamma_h \gamma_m)$

### Verification based on sectional forces

$\gamma_r S_d / R_d \leq 1.0$

$R(f_k) / S(f_k) \geq \gamma_c \gamma_h \gamma_m / \gamma_f \gamma_m$

### Consistency

$s = \gamma_r \gamma_h \gamma_m \gamma_f = 2.45 \rightarrow 2.5$

If $\gamma_f = 1.1$, $\gamma_h = 1.1$, $\gamma_m = 1.3$, $\gamma_n = 1.3$ and $\gamma_r = 1.2$

$\lambda_u = \frac{F_u}{F_k} \geq s$

**Figure 6.4:** Relationship between the load coefficient-based verification method and the sectional force-based verification method for renovation design of ageing sewers

(2) Safety factors used in performance evaluation by load coefficients

Table 6.5 shows the recommended values of safety factors that have been used for performance evaluation based on load coefficients for renovation design of ageing...
sewers. In determining these safety factors for the ultimate limit state, the emphasis is placed on safe design. Figure 6.5 shows the analysis procedure for evaluating the ultimate load-carrying capacity using the load coefficient.

### Table 6.5: Values of standard safety factors and safety factors used for renovation design of ageing sewers

<table>
<thead>
<tr>
<th>Safety factors</th>
<th>Material factor $\gamma_m$</th>
<th>Member factor $\gamma_s$</th>
<th>Structural analysis factor $\gamma_a$</th>
<th>Load factor $\gamma_i$</th>
<th>Structure factor $\gamma_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability limit state (serviceability)</td>
<td>Concrete $\gamma_c = 1.0$</td>
<td>Steel $\gamma_s = 1.0$</td>
<td>$\gamma_a = 1.0$</td>
<td>$\gamma_i = 1.0$</td>
<td>$\gamma_f = 1.0$</td>
</tr>
<tr>
<td>For composite pipe</td>
<td>$1.0$</td>
<td>$1.0$</td>
<td>$1.0$</td>
<td>$1.0$</td>
<td>$1.0$</td>
</tr>
<tr>
<td>Ultimate limit state (sectional failure)</td>
<td>$1.3$</td>
<td>$1.0$ or $1.05$</td>
<td>$1.1$ to $1.3$</td>
<td>$1.0$</td>
<td>$1.0$ to $1.2$</td>
</tr>
<tr>
<td>For composite pipe</td>
<td>$1.3$ (for both $\gamma_c$ and $\gamma_s$)</td>
<td>$1.3$</td>
<td>$1.1$</td>
<td>$1.2$</td>
<td>$1.1$</td>
</tr>
</tbody>
</table>

**Figure 6.5:** Flow of performance verification for renovation design of ageing sewers based on load coefficients (example: overburden thickness of 4 m or greater)
6.5 Performance Verification under Earthquake Loading

6.5.1 Seismic performance requirements

As set forth in the JSWA Seismic Guidelines, seismic performance requirements for renovated sewer pipes are classified according to the earthquake ground motions concerned. The earthquake resistance is evaluated according to the following verification criteria:

- Verification criterion for Level 1 earthquake ground motion
  The verification criterion corresponds to the serviceability limit state.

- Verification criterion for Level 2 earthquake ground motion
  The verification criterion corresponds to the ultimate limit state.

The seismic performance requirements specified in the JSCE Standard Specifications meet the requirements specified in the JSWA Seismic Guidelines. Current practice, therefore, is to use the verification indicators specified in the JSCE code. As shown in Table 6.6, in the seismic performance verification of a composite pipe in the cross-sectional direction, the occurrence of rebar yielding is used as the indicator for the serviceability limit state, and the ultimate displacement and the shear capacity are used as the indicators for the ultimate limit state. The flow of seismic performance verification for renovation design of ageing sewers is shown in Fig. 6.6.

![Flowchart](image)

**Figure 6.6:** Flow of verification on seismic performance for renovation design of ageing sewers
Table 6.6: Comparison of guidelines for performance requirements under earthquake loading

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1</td>
<td>Retaining design discharge capacity</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ability to maintain the discharge capacity shown on the flow calculation sheet</td>
<td>Level 1 seismic performance</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Able to remain in a functionally sound and usable condition even in the event of an earthquake</td>
<td>Post-earthquake residual displacement is sufficiently small. Steel reinforcement does not yield. Level 2 seismic performance</td>
<td></td>
</tr>
<tr>
<td>Level 2</td>
<td>Retaining discharge function</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Design discharge capacity is difficult to maintain, but the pipeline is still able to pass wastewater downstream until corrective measures such as repair and reconstruction are taken.</td>
<td>Functions can be restored shortly after an earthquake, and there is no need for structural reinforcement. The load-carrying capacity of the structure does not decrease because of an earthquake. Response displacement is within ultimate displacement. Shear failure does not occur.</td>
<td></td>
</tr>
</tbody>
</table>

Note: As a Level 3 seismic performance requirement under Level 2 earthquake ground motion, the JSCE code specifies the ability of a structure to withstand an earthquake without collapsing. From the viewpoint of the ability to maintain the discharge function, that can be regarded as a synonymous requirement. It was considered appropriate, however, to check on Level 2 seismic performance as shown above in order to be on the safe side.

In the longitudinal direction of the pipe, the basic rule is to make sure the lining members (push-fit connectors) in the pipe are joined together to be watertight. Common practice is to conduct experiments under the conditions described below to check the seismic performance.

1. In the verification of push-fit connector performance, checks are made to make sure push-fit joints remain connected and watertight in the event of pullout due to the permanent strain (1.5%) assumed in view of the data obtained from the Hyogoken Nanbu Earthquake.
2. In the verification of push-fit connector performance in the event of land subsidence due to liquefaction, checks are made to make sure push-fit joints remain connected and watertight in the event of deformation corresponding to an average span of 30 m and a subsidence of 30 cm.
6.5.2 Verification for serviceability limit state

The performance requirement in the serviceability limit state, which is the verification criterion for Level 1 earthquake ground motion, is the ability of a renovated sewer to remain fully functional without needing repair in the event of an earthquake. Therefore, checks are made to make sure the residual displacement of the structure after an earthquake is sufficiently small, and the criterion is the non-occurrence of rebar yielding.

6.5.3 Verification for ultimate limit state

The performance requirement in the ultimate limit state, which is the verification criterion for Level 2 earthquake ground motion, is the ability of a renovated sewer to restore functionality within a short period of time following an earthquake without needing structural reinforcement. Therefore, checks are made to make sure the load-carrying capacity of the structure does not decrease, that response displacement does not exceed the ultimate displacement and that shear failure does not occur.

Specifically, the criterion that response displacement does not exceed ultimate displacement can be checked by making sure that in-plane compressive strain does not reach or exceed two times the strain corresponding to the compressive strength. Specifically, the criterion that response displacement does not exceed ultimate displacement can be checked by making sure that in-plane compressive strain does not reach or exceed two times the strain corresponding to the compressive strength.

\[
\gamma_i \gamma_d \gamma_m(\text{response}) \varepsilon_c < 2 \varepsilon_{\text{peak}} / \gamma_b / \gamma_m(\text{limit})
\]  

where \( \varepsilon_c \): in-plane compressive strain; \( \varepsilon_{\text{peak}} \): strain corresponding to compressive strength (usually 0.002); \( \gamma_i \): member factor; \( \gamma_d \): structure factor; \( \gamma_a \): structural analysis factor; and \( \gamma_m \): material factor (response value: 1.0, limit value: 1.3). The non-occurrence of shear failure can be verified by verifying that in-plane shear force does not exceed the design shear capacity of the member.

\[
\gamma_i \gamma_d V < V_d
\]  

where \( V \): in-plane shear force and \( V_d \): design shear capacity.

6.5.4 Safety factors

The safety factors used in this study are shown in Table 6.7. Basically, they are in accordance with the JSCE Standard Specifications and have been set on the assumption that a nonlinear analysis method with fully proven accuracy is used. In the verification, it is desirable that different material factors and member factors be used for response analysis in the case where response displacement is calculated and
in the case where the non-occurrence of shear failure is verified. In practice, however, these two tasks are often performed in one response analysis with all safety factors being set at 1.0.

Safety factors (material factor $y_m$, member factor $y_b$, structural analysis factor $y_a$, load factor $y_f$ and structure factor $y_i$) for Grade 2 seismic performance required against Level 2 earthquake ground motion have been determined according to the concepts described below.

Table 6.7: Safety factors considered for seismic performance verification (JSCE, 2012)

<table>
<thead>
<tr>
<th>Safety factor</th>
<th>Material factor $y_m$</th>
<th>Member factor $y_b$</th>
<th>Structural analysis factor $y_a$</th>
<th>Load factor $y_f$</th>
<th>Structure factor $y_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic performance</td>
<td>Concrete 1.0</td>
<td>Steel 1.0</td>
<td>Concrete 1.0</td>
<td>Concrete 1.0</td>
<td>Concrete 1.0</td>
</tr>
<tr>
<td>Grade 1 seismic performance</td>
<td>Response value and limit value</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Grade 2 seismic performance</td>
<td>Response value</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0 (1.0–1.2)</td>
<td>1.0 (1.0–1.2)</td>
</tr>
<tr>
<td>Limit value</td>
<td>1.3</td>
<td>1.0</td>
<td>1.0</td>
<td>1.3 or 1.56** (1.1–1.3)</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* For displacement limit value
** For shear force calculation

1. Material factor $y_m$: The material factor is determined by taking into consideration factors such as a change in an undesirable direction from the characteristic value of material strength, differences in material properties between test specimens and structure materials, the influence of material properties on the limit state and changes over time in material properties. The response value and limit value used were 1.0 and 1.3, respectively.

2. Member factor $y_b$: It is generally known that the shear capacity of a structural member subjected to reversed cyclic loading decreases in the large deformation range, but the rate of such decrease has not yet been evaluated quantitatively. The JSCE Standard Specifications require that the shear force to be resisted by the concrete be increased by a factor of 1.2 or so in the shear capacity calculation formula (i.e. $1.3 \times 1.2 = 1.56$). The limit value, therefore, is 1.56 in the case where flexural yielding occurs and 1.3 in the case where flexural yielding does not occur.

3. Structural analysis factor $y_a$: The structural analysis factor is 1.0.

4. Load factor $y_f$: The load factor is 1.0.

5. Structure factor $y_i$: The structure factor is 1.0.
6.5.5 Analysis method used for verification

Records of observation of underground pipelines, immersed tube tunnels and other underground structures during earthquakes and the results of model vibration experiments have shown that earthquake-induced vibration properties of underground structures have the following characteristics:

1. The apparent unit weight of underground structures is smaller than or similar to that of the surrounding ground. Therefore, during an earthquake, underground structures do not vibrate differently, but vibrate with the surrounding ground.
2. Seismically induced structural behaviour is governed not by seismic inertia force but by the relative displacement of the surrounding ground (ground strain).

Hence, methods for calculating the earthquake resistance of underground structures have been developed on the basis of the abovementioned vibration characteristics. Those methods can be broadly classified into static analysis methods and dynamic analysis methods.

Static analysis methods include the response displacement method and the response seismic coefficient method. In these methods, seismically induced dynamic external forces are replaced with static forces, and structural behaviour under earthquake loading (e.g. response displacement, seismically induced stress) is calculated by applying those forces on the structure of interest.

In dynamic analysis methods, the object to be analysed is modelled as a dynamic mechanics model with dynamic response characteristics, and the time history response of the structure of interest is calculated by directly inputting the time history waveform of ground motion. The response behaviour of an underground structure can be determined by solving an equation of motion formed by taking into account its mass, stiffness and damping characteristics.

The JSWA Seismic Guidelines describe a number of methodologies for evaluating the earthquake resistance of existing pipes and renovated pipes. Among those methods, the Tokyo Metropolitan Government has adopted a verification method based on nonlinear dynamic response analysis, aiming at accurately reflecting the condition of buried pipes. This verification method is discussed in the following section.

6.5.6 Earthquake resistance verification based on nonlinear dynamic analysis

(1) Overview of the nonlinear dynamic analysis method
The dynamic response of a structure subjected to earthquake ground motion varies depending on mass, stiffness and damping. This section briefly describes the time history response analysis method as a technique for analysing the response of a vibrating system to dynamic disturbances such as earthquake ground motion.
The equation of motion of a single-degree-of-freedom elastoplastic system can be expressed as shown below. If given the input $-m\ddot{x}$ at a given time, this equation of motion can be solved by using direct integration.

$$m\ddot{x} + c\dot{x} + Q(x) = -m\ddot{x}$$  \hspace{1cm} (6.9)

where $m$: mass; $c$: damping factor; $x$: relative displacement; and $\ddot{x}$: the acceleration of ground motion. $Q(x)$, called the restoring force characteristic, is a function of the deformation history. Within the small structural deformation range, structural behaviour can be deemed more or less elastic. As deformation increases, however, damage resulting from cracking, plasticisation, etc. accumulates locally so that the restoring force–deformation curves form hysteresis loops. When trying to determine the response properties of a structure through nonlinear dynamic analysis, it is important to set such elastoplastic restoring force characteristics or hysteresis characteristics appropriately.

Various nonlinear hysteresis models for nonlinear dynamic analysis have been proposed for use under different material and structural conditions. Figure 6.7 shows three types of basic restoring force model. Type (a), the bilinear model, is widely used as a model of the characteristics of a highly deformable (i.e. tough) structure. A model of this type is often used to model a steel structure, and is characterised by relatively large energy consumption due to the hysteresis effect. Type (b), the stiffness reduction type model, does not follow the same loops and shows gradual stiffness reduction after the occurrence of plastic deformation. A model of this type is often used to model a reinforced concrete structure. Type (c), the slip type model, shows low unloading stiffness in the small restoring force range and is characterised by relatively small energy consumption due to the hysteresis effect. Thus, if a nonlinear hysteresis model to be used is to be determined, its skeleton curve and hysteresis rule are necessary. Since an ageing sewer to be renovated is a reinforced concrete structure, a Type (b) model is used for nonlinear hysteresis modelling of a composite pipe.

![Figure 6.7: Examples of restoring force models: (a) bilinear type; (b) reduced stiffness type; (c) slip type.](image-url)
(2) Structural analysis model
In view of the nonlinearity of materials, the basic rule is to perform verification on the basis of response values obtained through time history response analysis. Another rule is to conduct structure–soil coupling analysis by using reliable nonlinear hysteresis models of materials and members. Cracking of concrete is modelled by using a smeared crack model. For dynamic modelling, path-dependent hysteresis models are used for both concrete and reinforcing steel. For ground modelling, the nonlinearity of ground during a large-scale earthquake is considered. As an example, UC-win/WCOMD, a commercial programme that meets the requirements mentioned above, is selected for dynamic response analysis (Okamura and Maekawa, 1991; FORUM 8, 2013).

In cases where a coupling analysis of a soil–structure system is conducted, the vibration of the structure and the energy of scattered waves generated by irregularities of the ground are confined in the system. It is therefore necessary to use a finite element model covering a sufficiently large area of ground. It is also necessary to use artificially introduced imaginary boundaries to absorb wave energy. Such boundaries that are widely used include viscous boundaries, energy-transmitting boundaries and superposition boundaries. Superposition boundaries, which are used in UC-win/WCOMD, are briefly explained below.

When a scattering wave reaches an artificial boundary, the wave is reflected in the same phase if the boundary is free. If the boundary is fixed, the wave is reflected in the opposite phase. The reflected wave can then be eliminated by adding the two waves together. If, however, the reflection is repeated at two or more boundaries, the reflected waves cannot be eliminated. Another drawback of the superposition boundary approach is that the entire region needs to be analysed. In view of these problems, Cundall et al. proposed a method of superimposing the solutions obtained from constant-velocity and constant-strain boundary conditions, in place of fixed and free boundaries (Cundall, Kunar, Carpenter, et al., 1978). In the proposed method, waves can be cancelled in near-boundary zones. Since two-layer finite elements are defined only in the near-boundary zones as shown in Fig. 6.8, reflected waves can be cancelled with good accuracy with only a small number of degrees of freedom.

(3) Limit values in verification
Limit values used in the verification for Level 2 earthquake ground motion are as follows. In the verification for ultimate displacement, based on Eq. (6.7) a limit value of 0.004, which is two times the strain corresponding to typical concrete compressive strength (0.002), is used. In shear-related verification, design shear capacity is calculated according to the design shear capacity formula for linear members (JSCE, 2012):

\[ V_{yd} = V_{cd} + V_{sd} + V_{ped} \]  

(6.10)
where \( V_{cd} \) is the design shear capacity of a linear member without shear reinforcement, calculated as

\[
V_{cd} = \beta_d \cdot \beta_p \cdot \beta_n \cdot f_{vcd} \cdot b_w \cdot d / \gamma_b
\]

and

\[
f_{vcd} = 0.20 \sqrt{f'_{cd} (N/mm^2)} \text{ where } f_{vcd} \leq 0.72 (N/mm^2)
\]

where

- \( \beta_d = \frac{4}{\sqrt{1/d (d:m)}} \) where 1.5 is used if \( \beta_d > 1.5 \)
- \( \beta_p = \frac{3}{100P_v} \) where 1.5 is used if \( \beta_p > 1.5 \)
- \( \beta_n = 1 + 2M_0 / M_{ud} \) (if \( N_d > 0 \)) where 2 is used if \( \beta_n > 2 \)

or

\[
= 1 + 4M_0 / M_{ud} \text{ (if } N_d < 0 \text{)} \text{ and } \beta_n = 0 \text{ if } \beta_n > 0
\]

where \( N_d \): design compressive force; \( M_{ud} \): pure flexural capacity; \( M_0 \): bending moment necessary to cancel stress caused by axial force at extreme tension fibre corresponding to design bending moment \( M_d \); \( b_w \): midsection width; \( d \): effective height; \( P_v = A_s / (b_w \cdot d) \); \( A_s \): cross-sectional area of steel in tension zone; \( f'_{cd} \): design compressive strength of concrete, in N/mm²; \( \gamma_b \): usually, 1.3 may be used (1.6 if flexural yielding occurs); \( V_{sd} \): design shear capacity provided by shear reinforcement; and \( V_{ped} \): shear component of effective tension in axial tendon.

The formula shown above cannot be applied directly to lining members because they are composite members. It is therefore necessary to make effective use of the
advantages of each renovation construction method and to evaluate shear capacity on the basis of relevant test results. For example, for the SPR method, the shear strength of lining members shown in Table 3.35 is calculated from the results of single shear tests conducted on composite members, and shear capacity is calculated on the basis of Eq. (6.13) and Eq. (6.14):

\[
V_{spr} = \sigma_{spr} H
\]  

(6.13)

where \(V_{spr}\) : shear capacity of SPR lining member (contribution by SPR lining member); \(\sigma_{spr}\) : shear strength of SPR lining member (from Table 3.35); and \(H\): thickness of SPR lining member. From the above, the design shear capacity of an SPR composite pipe can be calculated from:

\[
V_d = V_{cd} + V_{spr} \gamma_m \gamma_b
\]  

(6.14)

where \(V_d\): design shear capacity of SPR composite cross section; \(V_{cd}\): design shear capacity of linear member without shear reinforcement; and \(V_{spr}\): shear capacity of a SPR lining member.

(4) Ground motions considered

The ground motions used to verify the earthquake resistance of renovated sewers are Level 1 and Level 2 earthquake ground motions. The correct way in this case would be to decide on ground motions to be considered in view of factors such as the seismic activity level at and near the construction site, earthquake source characteristics and characteristics of the propagation of ground motion from the source to the construction site. Since, however, deriving time history acceleration waveforms by this approach would be laborious, guidelines such as the JSCE Standard Specifications and the Specifications for Highway Bridges (JRA, 2012) allow the use of simulated ground motions containing vibration components that increase the influence on the structure. Those guidelines state that for Level 2 earthquake ground motion, it is necessary to take into consideration two types of ground motion: inland near-field and oceanic/interplate.

When conducting seismic verification, the Tokyo Metropolitan Government uses the simulated time history waveforms of ground motion shown in the JSCE Standard Specifications. The JSCE code shows acceleration response spectra to specify earthquake loads and gives four sample waveforms of Level 2 earthquake ground motion as time history waveforms, as shown in Fig. 6.9. Figure 6.10 shows the acceleration response spectra corresponding to those seismic waveforms. The following four sample seismic waveforms were derived:

1. The “inland type 1” earthquake ground motion waveform was defined on the basis of a past inland strong motion record.
2. The “inland type 2” earthquake ground motion waveform was taken from the Kobe Port Island strong-motion seismograph record of the Hyogoken Nanbu Earthquake of 1995.
Figure 6.9: Simulated ground motion waveforms (JSCE, 2012)
3. The “interplate (oceanic) type 1” earthquake ground motion waveform was obtained by making corrections to an observation record.

4. The “interplate (oceanic) type 2” earthquake ground motion waveform was derived by calculation from the fault model of the scenario Tokai Earthquake proposed by the Central Disaster Management Council.

![Figure 6.10: Acceleration response spectra of inland and oceanic earthquakes (damping factor = 5%)](JSCE, 2012)

(5) Ground conditions

Ground conditions are extracted from available records of boring surveys conducted near the representative cross section of the sewer line to be renovated. The bedrock surface assumed, at which ground motion is input, must be the upper surface of a ground layer that is as extensive as the project site, is firm and has sufficiently higher shear wave velocity compared with the ground surface. The bedrock surface must
be selected by collecting not only geological survey data obtained at the project site but also information on the surrounding areas and comprehensively evaluating the information thus obtained.

The bedrock surface may be chosen as the upper surface of a continuous layer having an N-value (SPT blow count) of 50 or more and a shear wave velocity $V_s$ of not lower than about 300 m/s. If detailed geotechnical survey results are available, a bedrock surface can be determined appropriately. It is therefore necessary to take site-specific ground conditions into consideration.

### 6.5.7 Earthquake resistance verification based on response displacement method

1. **Position of response displacement method**
   The JSWA Seismic Guidelines state that it is good practice to consider the use of a more accurate method such as nonlinear dynamic analysis that takes into consideration the nonlinearity of ground and structural members, but it also permits the use of the response displacement method in view of its simplicity in engineering practice. Although verification by the response displacement method has not been recognised as a sufficiently rational verification approach applicable to renovation design of ageing sewers, it has been widely used in the seismic design of ordinary underground structures and sewer systems. Therefore, the response displacement method will also be used for calculating seismically induced loads. Concepts concerning the use of the response displacement method for earthquake resistance verification are described in the subsequent sections.

2. **Earthquake-induced ground displacement**
   1) Calculation of ground displacement based on design response velocity spectrum
   The JSWA Seismic Guidelines and the JSWA Guidelines show response velocity spectra for Level 1 and Level 2 earthquake ground motions for use at the design stage. The response displacement of the surface layer of ground is calculated by the so-called response spectrum method on the basis of the natural period of ground. Types of ground motion, however, are not distinguished.

   In the response displacement method, the influence of ground motion is applied as an external force calculated from the response displacement of ground, so the relations concerning design ground motion and the response of ground are important. Basically, the methods shown in the JSWA Seismic Guidelines and the JSWA Guidelines assume that stiffness is uniform in the depth direction and that the first mode of shear vibration is the only response vibration mode. Therefore, the response displacement of the surface layer of ground can be calculated by using a simple formula:

   $$ U_h(z) = \frac{2}{\pi^2} \cdot S_v \cdot T_s \cdot \cos \left( \frac{\pi z}{2H} \right) $$

   (6.15)
where \( U_h(z) \): displacement amplitude in horizontal direction at depth \( z \) from ground surface; \( S_v \): design response velocity; \( T_s \): natural period of surface layer of ground; and \( H \): thickness of surface layer of ground. Figures 6.11 and 6.12 show response velocity spectra for design use, and Fig. 6.13 shows the seismic areal-division map of Japan for classification of seismic areas.
2) Calculation of ground displacement based on ground response analysis
Sewer pipes are often buried in multi-layer ground, and the influence of similar design response spectra on a structure varies depending on the type of ground motion. If, therefore, seismic performance verification is to be made with higher accuracy, it is desirable that the structural response be calculated by inputting the ground response calculated through a time history response analysis of ground alone. The seismic behaviour of an underground structure is greatly affected by the rate of change in horizontal displacement in the vertical direction, instead of the magnitude of absolute displacement of ground at the location of the structure. Thus, the displacement used in the response displacement method is the displacement of ground at the time when the relative displacement of ground between the upper and lower ends of the structure is maximised.

The equivalent linearisation method, a method of response analysis in the frequency domain, is widely used for analysing the time history response of ground. The equivalent linearisation method is used in cases where the shear strain amplitude of ground during an earthquake is relatively small (about $10^{-3}$ or less). An equivalent linear model expresses soil as a visco-elastic body with stiffness and viscosity and sets equivalent shear moduli and viscous damping factors during the duration of ground motion according to the amplitude of earthquake-induced shear strain (effective strain). Linear analyses are repeated by using the equivalent linear model until the
shear strain amplitude converges. This method is practical and useful because stable solutions can be obtained and the calculation time is short.

(3) Inertia force
Since pipeline facilities are usually lighter than the soil of the same cross-sectional area, inertia force is ignored in many cases. For renovated pipes, however, it was decided to take inertia force into consideration to make the renovation design of ageing sewers consistent with the design of other civil engineering structures. As a basic rule, inertia force was calculated by multiplying the weight of the structure by the design seismic coefficient. In FEM analysis, inertia force was made to act as an external force by multiplying each nodal force corresponding to self-weight by the design horizontal seismic coefficient. The design seismic coefficient was determined in accordance with the JSWA Seismic Guidelines. The influence of the design vertical seismic coefficient is often ignored because it is usually small.

(4) Soil springs
Soil springs defined along the perimeter of the structure of interest in the response displacement method are positioned normal and tangential to the outer circumference of the pipe. Spring values are determined from moduli of soil reaction, taking into account node-dominated areas in the FEM analysis model. There are a number of methods for defining soil springs, and it was decided to use the method employed in the JSWA Examples of Seismic Design Calculation for Sewer Facilities (JSWA, 2001). Although there are various approaches to dealing with loading width, it was decided, in principle, to use product length for ready-made products such as circular pipes, precast box culverts and arch culverts. Cast-in-place pipes require separate definitions. For example, a single span length may be used for tunnels.

(5) Earthquake-induced skin shear force
The outer surface of an underground structure in contact with the surrounding soil is acted upon by skin shear force in the event of an earthquake, and so such skin shear force is taken into consideration. Although there are cases in which earthquake-induced skin shear force is ignored for circular pipes, in view of common practice, such skin shear force should be taken into consideration. A commonly used method is to calculate earthquake-induced skin shear force by using the equation shown below on the basis of the design response velocity spectrum. Another common approach is to use the shear force at the time when the maximum ground displacement occurs obtained from one-dimensional ground response analysis.

\[ \tau = \frac{G_d}{\pi H} \cdot S_v \cdot T_s \cdot \sin \left( \frac{\pi \xi}{2H} \right) \]  

(6.16)
(6) Verification method based on nonlinear FEM analysis

Figures 6.14 and 6.15 show the flows of analysis and verification. For Level 1 earthquake ground motion, seismically induced external force is increased and made to act in the initial stress state under normal loading (dead load + static earth pressure + overburden load) to confirm that steel reinforcement does not yield or cracking does not occur until the design ground motion is reached.

<table>
<thead>
<tr>
<th>Performance verification</th>
<th>Seismic diagnosis of existing pipe: Determine if seismic retrofit is necessary.</th>
<th>Seismic diagnosis of renovated pipe: renovation design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose of verification</td>
<td>Level 1 earthquake ground motion (serviceability limit state)</td>
<td>Steel reinforcement should not yield (or cracking does not occur).</td>
</tr>
</tbody>
</table>

Figure 6.14: Flow of performance verification under Level 1 earthquake ground motion
Performance Verification under Earthquake Loading

Figure 6.15: Flow of performance verification under Level 2 earthquake ground motion

For Level 2 earthquake ground motion, seismically induced external force is increased and made to act in the initial stress state under normal loading (dead load + static earth pressure + overburden load) and the applied load is increased until the ultimate state is reached. In the verification concerning the ultimate limit state, the design sectional capacity and design sectional force used for verification are calculated as follows:
1. Calculation of the maximum sectional force (= design sectional capacity) at ultimate failure and determination of the location where that occurs
2. Calculation of sectional force (= design sectional force) at the abovementioned location under Level 2 earthquake ground motion

Design sectional force is calculated by applying the specified safety factor to the sectional force calculated from the stress state under design Level 2 earthquake ground motion. Design sectional capacity is calculated by applying the specified safety factor to the maximum sectional force that has occurred at or before failure. In safety verification, checks are made to confirm that design sectional force does not exceed design sectional capacity.

References