Invited Paper

Snapback Failure Analysis for Large Scale Concrete Structures and its Application to Shear Capacity Study of Columns

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Abstract

A comprehensive failure analysis method for large-scale concrete structures is presented, which includes snap-back failure. Energy observation is used to describe the macroscopic mechanism of snapback occurrences and other softening failure modes.

The typical snapback failures of beams and columns by shear are numerically obtained using the arc length method with selected displacement control points. The fundamental understanding of the phenomena is extended to the failure mechanism of a column where shear capacity is affected by axial force.

1. Introduction

Concrete structures are generally designed so that their final failure is ductile to avoid unforeseen failure. However, this is not always possible because there are a variety of structural functions and dimensions are often determined for a mix of reasons, which sometimes preclude such safety criteria.

As an example, let us consider large-scale short bridge piers. Such piers do not often require any lateral reinforcement since required horizontal shear capacity is satisfied by concrete portion alone. Even if they need reinforcements, the reinforcements are to satisfy the minimum reinforcement requirement. In a dam construction, what is needed is a mass and weight or a volume. The strength of the dam is satisfactory in most cases without any steel reinforcement. Therefore, if those structures fail, very brittle failures are anticipated. Such tendencies grow more prominent with increases in structural dimensions; in other words, the bigger a structure is, the greater is the tendency for that structure to fail in a brittle manner.

This paper aims at presenting failure mode analysis including brittle failure modes such as snap back with the extended arc length method. Energy criteria will be introduced to explain general snapback phenomena. The discussion of the snapback analysis will be extended to column failure by horizontal shear.

2. Energy criteria to determine snapback mode of failure

Let us examine the post peak behavior of a structure, whose load deflection curve may be shown with post peak softening branches as illustrated in Fig. 1.

Three cases are shown; one is a softening branch, another one is a branch that falls vertically, and the last one is snapback. These characteristics are governed by the relative magnitude of energy absorption and release ability in different parts of a structure under decreasing load. The area surrounded by the vertical line and the softening branch, A, shows that with the amount shown by the area, the energy absorbing capacity of a structure is larger than the energy releasing ability during load decrease. Amount A is equal to the energy given to a structure by decreasing external load during deformation.

$$A = E_{\text{absorption}} - E_{\text{release}}$$

On the other hand, snapback behavior is characterized by the fact that the energy absorbing capacity is smaller

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than the energy release by the amount shown with the area surrounded by the snapback branch and the vertical line, B.

\[ B = - E_{\text{absorption}} + E_{\text{release}} \]  

(2)

where B is the amount of energy that a structure will exert on the external load, and if no external load does exist, it is converted to dynamic energy such as self vibration.

If the energy absorbing capacity is equal to the energy release amount, a load deflection curve then follows the vertical line and the structure does not exert any work on the external load and nor does the external load exert any work on the structure.

In the case of snap back shear failure of reinforced concrete beams, energy release is mainly due to flexural rebound at the span center while energy absorption is mainly caused by diagonal tension deformation and shear deformation in a shear span.

For flexural failure of pre-stressed concrete members, snapback is recognized when compressive failure in the upper compression zone occurs. This phenomenon is due to the fact that the localized compression zone can hardly absorb the energy released from both unloading pre-stressing tendons and the compressive zone of concrete outside the localized zone.

In general, a failure is always accompanied by the localization of strain where loading is taking place. However, it also accompanies the unloading zone where strain is decreasing. Therefore, the failure pattern of a structure can be effectively identified tracing the energy absorbing area during load decrease.

2.1. Uniaxial column analysis

Let us consider a uniform bar loaded under tension and divided into m elements as shown in Fig. 2. Prior to reaching the tensile strength \( f_t \), the strain distributes uniformly. A linear relation is assumed between the normal stress and the normal strain \( \varepsilon \):

\[ \sigma = E \varepsilon \]  

(3)

with E being Young’s modulus. After reaching the peak strength, descending slope \( h \) defines the softening slope of the concrete under tension. In the post peak regime, the constitutive equation can thus be written as:

\[ \sigma = f_t + h(\varepsilon - \varepsilon_0) \]  

(4)

where \( h \) may be termed as the softening modulus and \( h < 0 \). For linear softening, we have:

\[ h = - \frac{f_t}{(\varepsilon_u - \varepsilon_0)} \]  

(5)

where \( \varepsilon_u \) denotes the strain where concrete exhausts its loading capacity.

Assuming that one element has tensile strength marginally below that of the other \( (m-1) \) elements, the column will undergo deformation such that the one element

![Diagram](image-url)
elongates or is loaded while the other (m-1) elements contract or are unloaded when the external load is close to the maximum loading point (refer to Fig. 2 (c)). Therefore, energy is absorbed in one element and energy is released in the other element. Energy absorption amount \( \Delta E_{\text{abs}} \) in the element is calculated as Equation (6) assuming unit volume for all elements.

\[
\Delta E_{\text{abs}} = \frac{1}{2} (\varepsilon - \varepsilon_0) \{ 2 f + h(\varepsilon - \varepsilon_0) \} \tag{6}
\]

while energy release amount \( \Delta E_{\text{releas}} \) is calculated as

\[
\Delta E_{\text{releas}} = (m-1) \left( \frac{1}{2} f^2 - \frac{1}{2} \sigma^2 \right) \tag{7}
\]

The condition under which these values are equal is:

\[
h = -\frac{E}{(m-1)} \tag{8}
\]

In this condition, the total elongation of one element is equal to the total contraction of the (m-1) elements and external load does not exert work.

However, if \( h \leq -\frac{E}{(m-1)} \), then snapback occurs due to the larger contraction compared with elongation. The opposite occurs if \( h > -\frac{E}{(m-1)} \).

In other words, snapback or neutral deformation occurs when

\[
\Delta E_{\text{abs}} \leq \Delta E_{\text{releas}} \tag{9}
\]

and normal ductile deformation occurs if

\[
\Delta E_{\text{abs}} > \Delta E_{\text{releas}} \tag{10}
\]

3. Energy criteria to determine snapback mode of failure for general cases

Whether snap back or ductile failure may occur is not overly explicit in a general case such as a simple column. In such general cases, step by step load increment or displacement increment analysis should be performed. However, these single single-parameter analyses fail when they encounter snapback instability. For general use, an arc length analysis with selected control points should be adopted. This section outlines this method.

Numerical analysis of the snapback failure mode is difficult due to the convergence problems that occur after the peak load. In these situations, the arc length algorithm with selected displacement controlling points method (Tanabe and Itoh 2001) is powerful. This method is similar to the indirect displacement control proposed by deBorst and others. In these calculations, the importance of specifying a few appropriate displacement-controlling points in order to assure convergence is emphasized.

3.1. Arc length formulation for non-proportional and alternating loading

Usually the arc length formulation is written for a proportional loading case. Here, mixed non-proportional loading cases as well as alternating loading cases are included following Batos and Dhatt (Batos and Dhatt 1976).

The formulation is expressed as

\[
\Delta [\{P\}] = \Delta \lambda_i \{P_{fix}\} \tag{11}
\]

where \( \Delta \lambda_i \) = incremental load parameter and \( \{P_{fix}\} \) = fixed load increment vector.

Subscript index \( i \) denotes the \( i \)-th iterated values within a loading step (n+1). The force equilibrium is expressed as

\[
\{g^{n+1}\} = \{P_{fix}\} + \{P^{n+1}\} - \{R^{n+1}\} = 0 \tag{12}
\]

where

\[
\{P_{fix}\} = \text{Fixed load vector} \\
\{g^{n+1}\} = \text{Unbalanced force vector at (n+1) step of load increments} \\
\{P^{n+1}\} = \text{External proportional load vector at (n+1) step} \\
\{R^{n+1}\} = \text{Equivalent nodal force at (n+1) step loading}
\]

Substituting Equation (11) to Equation (12),

\[
\{g^{n+1}\} = \lambda_i^{n+1} \{P_{fix}\} - \{R_j^{n+1}\} = 0 \tag{13}
\]

where

\[
\lambda_i^{n+1} = \lambda^n + \sum_{j=1}^{i-1} \delta \lambda_j^{n+1} + \delta \lambda_i = \lambda_{i-1}^{n+1} + \delta \lambda_i \tag{14}
\]

\[
\{R_j^{n+1}\} = \{R_j^{n+1}\} + [K_{fix}] \delta \{U_j\} \tag{15}
\]

where

\[
[K_{fix}] = \text{Initial elastic stiffness matrix} \\
\{U\} = \text{nodal displacement vector} \\
\delta = \text{Increment during an iteration within a load step}
\]

Substituting Equations (14) and (15), Equation (13) can be rewritten as

\[
\{g^{n+1}\} = \lambda_i^{n+1} \{P_{fix}\} + \delta \lambda_i \{P_{fix}\} \\
-\{R_j^{n+1}\} - [K_{fix}] \delta \{U_j\} = 0 \tag{16}
\]

Therefore, modification vector \( \delta \{U_j\} \) is obtained as
\[ \delta \{ U \} = \{ K \}_{fix}^{-1} (\lambda^{n+1} \{ P \}_{fix} - \{ R \}_{n+1}) + \delta \lambda \{ K \}_{fix}^{-1} \{ P \}_{fix} \]
\[ \delta \{ U \} = \delta \{ \bar{U} \} + \delta \lambda \delta \{ U \} \] (17)
\[ \delta \{ \bar{U} \} = \{ K \}_{fix}^{-1} (\lambda^{n+1} \{ P \}_{fix} - \{ R \}_{n+1}) \]
\[ \delta \{ U \} = \{ K \}_{fix}^{-1} \{ P \}_{fix} \]

in which \( \{ K \}_{fix} \) is kept constant during iteration.

In the procedure, unknown parameter \( \delta \lambda \) is involved which is to satisfy the arc length constraint, Equation (18).

\[ \Delta \{ U \}^T \Delta \{ U \} + \Delta \lambda^2 \varphi^2 \{ P \}_{fix}^T \{ P \}_{fix} = \Delta l^2 \] (18)

where \( \Delta \{ U \} = \{ U^{n+1} \} - \{ U^n \} \), and \( \varphi = \) scaling parameter and \( \Delta l = \) arc length.

\[ \Delta \{ U \} = \Delta \{ U_{n+1} \} + \delta \{ U \} \]
\[ = \Delta \{ U_{n+1} \} + \delta \{ \bar{U} \} + \delta \lambda \delta \{ U \} \] (19)
\[ \Delta \lambda = \Delta \lambda_{n+1} + \delta \lambda \] (20)

Substituting Equations (19) and (20) into Equation (18), we obtain the quadratic equation as

\[ a \delta \lambda^2 + b \delta \lambda + c = 0 \] (21)

\[ a = \delta \{ U \}_{1}^T \delta \{ U \}_{1} + \varphi^2 \{ P \}_{fix}^T \{ P \}_{fix} \]
\[ b = 2 \delta \{ U \}_{1}^T (\Delta \{ U_{n+1} \} + \delta \{ \bar{U} \}) + 2 \Delta \lambda_{n+1} - \varphi^2 \{ P \}_{fix}^T \{ P \}_{fix} \]
\[ c = (\Delta \{ U_{n+1} \} + \delta \{ \bar{U} \})^T (\Delta \{ U_{n+1} \} + \delta \{ \bar{U} \}) + \Delta \lambda^2 \varphi^2 \{ P \}_{fix}^T \{ P \}_{fix} - \Delta l^2 \]

Solving Equation (21), \( \delta \lambda \) can be obtained. In this calculation, \( \varphi = 0 \) is adopted as the cylindrical arc-length method.

### 3.2. Alternating load reversal

For this case, the sign in front of \( \Delta \lambda \) in Equations (11) is simply taken as a plus or minus sign depending on load reversal and exactly the same procedures as Equation (15) to Equation (17) is iterated until convergence is attained.

### 3.3 Appropriate selection of displacement path

Solving Equation (21), two different values for \( \delta \lambda \) are obtained and the proper selection of a value from the two is important.

The two values \( \Delta \{ U_{1} \} \), \( \Delta \{ U_{2} \} \) are expressed as

\[ \Delta \{ U_{1} \} = \Delta \{ U_{n+1} \} + \delta \{ \bar{U} \} + \delta \lambda \delta \{ U \} \]
\[ \Delta \{ U_{2} \} = \Delta \{ U_{n+1} \} + \delta \{ \bar{U} \} + 2 \delta \lambda \delta \{ U \} \]

and the one that produces a smaller angle with immediate past displacement increment \( \Delta \{ U_{n+1} \} \), is adopted, as schematically shown in Fig. 3. The other problem of numerical analysis in the softening region and in the snapback region is stable path tracing at bifurcation points. The conventional method to achieve this is that mentioned by Bazant (Bazant 1991), which consists in detecting bifurcation points and selecting a path that minimizes the second order work. Though we did not explicitly examine these characteristics at every calculation step in the following examples, we did use this method for the selected example and were able to confirm the attainment of a stable path or nearly stable path.

### 3.4. Selection of displacement controlling points

The generic arc length algorithm incorporates all the displacement components in the calculation of Equation (18). However, convergence often worsens as a result in the case of strong nonlinear problems in reinforced concrete structures with snapback instability. The remedy for those cases is known to adopt selected displacement controlling points instead of all the nodal displacements. In that case, it is far more important to select the points that sensitively affect the total convergence. Therefore, expressing a selected displacement vector as \( \Delta \{ U \}_x \), the arc length is written as

\[ \Delta \{ U \}_x \cdot \Delta \{ U \}_x = \Delta l^2 \] (24)

In view of Equation (17), \( \delta \lambda \) must satisfy the following equations

\[ a \delta \lambda^2 + b \delta \lambda + c = 0 \] (25)

\[ a = \delta \{ U \}_{1, x} \delta \{ U \}_{1, x} \]
\[ b = 2 \delta \{ U \}_{1, x} \Delta \{ U_{n+1, x} \} + \delta \{ \bar{U} \}_{1, x} \]
\[ c = (\Delta \{ U_{n+1, x} \} + \delta \{ \bar{U} \}_{n+1, x})^T (\Delta \{ U_{n+1, x} \} + \delta \{ \bar{U} \}_{n+1, x}) - \Delta l^2 \]
Fig. 4 Test specimen.

Fig. 5 S1 test results.

Fig. 6 S2 test results.

Fig. 7 S3 test results.

Fig. 8 S4 test results.

Fig. 9 Finite element mesh discretization.

Fig. 10 Comparisons of numerical and experimental results for S4.
In the snapback calculation, the calculation controlling point is adopted as shown in Fig. 9. Good results could not be obtained by taking controlling points elsewhere.

3.5. Failure pattern identification
When a post-peak load deflection relation of a structure is obtainable, the failure pattern which pertains to the post peak deformation becomes clearer tracing areas where energy absorption occurs and where energy release occurs in a structure.

Connection of energy absorbing areas automatically reveals where the strain localization occurs and accordingly where failure is progressing.
4. Snap-back shear failing beam analysis

For practical application of the method, the experiment by Y. Uchida at Gifu University is adopted and analyzed. The LECOM, the lattice equivalent continuum model Code (Itoh, Kongkeo, Nakamura and Tanabe 2002), developed at Nagoya University Concrete Laboratory is used to analyze the beam specimens. The parameter of LECOM Code used is described in each calculation.

4.1. Experimental report by Y. Uchida of snap-back failure of reinforced concrete beams

There have been very few experiments of shear failing beams in the past where snapback behavior was captured. However, Y. Uchida, et al. of Gifu University (Uchida 2003) recently successfully accomplished this by controlling the dissipating energy through skilful opening and closing of a load control valve in the experimental apparatus.

The specimens are shown in Fig. 4, with sectional...
dimensioning. Four specimens are used, all with the same dimensions, i.e., 100 mm by 200 mm section with a shear span of 300 mm (a/d=1.7) at one side of the beam and 500 mm at the other side (a/d=2.9), which is the experimentally targeted part. No shear reinforcements exist in the targeted span, while D6@100 shear reinforcement is placed in the other shear span to prevent failure in the span. Longitudinal reinforcements of 2.31% with D16, is placed. The compressive strength of the concrete is 54 MPa, the tensile strength is 3.96 MPa, and the fracture energy is 18.7 N/m.

Out of the four specimens, specimen S1 was not properly controlled and failed without post peak data, however, its crack pattern was very similar to the specimen S3 and it is estimated that the results could possibly have been similar to those for specimen S3. Specimen S2 failed by flexure, experiencing yielding of the longitudinal reinforcement and exhibiting a certain level of ductility. Experimental data for specimen S3 and specimen S4 successfully demonstrated the snapback
behavior in the post peak region. S3 was reported to have failed by shear compression while specimen S4 was reported to have failed by diagonal tension. These post peak load deflection curves are shown in Fig. 5 to Fig. 8 with their crack pattern. The scattering of the results is attributed to the keen sensitivity of the boundary criteria of the shear failure pattern at the shear span by depth ratio of 2.9, thus inducing scattering of the location of the first diagonal tension crack. If this occurs in the shear span close to the loading point, further extension of the crack is apt to grow in the compression zone where further crack extension to the top surface of the beam is hindered and direct diagonal tension failure is blocked. On the other hand, if the first diagonal crack starts from somewhere off from the loading point, then the crack can extend to the top surface without blockage by compression zone and brittle diagonal tension failure is apt to occur. In the following numerical analysis, this randomness of tensile strength of concrete is not treated and deterministic calculation is carried out focusing on specimens S4 and S3. The LECOM parameters for the shear lattices used were $E^* / E_c$, $\theta = \pi / 6$ and $\alpha = 0.58 \times 10^{-2}$ cm (Itoh, Kongkeo, Nakamura and Tanabe 2002).

Analysis was performed and a converged solution was obtained selecting one displacement control point solely at the mid-shear span and lower portion of the beam as shown in Fig. 9, something which is difficult to obtain by selecting arbitrary controlling points. The calculated load displacement curve is shown in Fig. 10 with the experimental curve of S4. Clear snapback after the peak point which is very close to the experimental curve can be seen. Examining the stress loading and the stress unloading zones when both the external load and the displacement are decreasing, the failure location during snapback is clearly visible. The figures show the energy absorptions and releases in terms of compression stress, tensile stress and shear stress. The definition of stress component in terms of the main lattice direction in the LECOM formulation is adopted in this paper, the details of which may be referred to (Itoh, Kongkeo, Nakamura and Tanabe 2002).

In the pre-peak region, the red colored portion of the line in Fig. 11 (a), most of the energy is absorbed by flexure at the center and by shear in the diagonal area.

![Graph](image)

**Table 1 Reinforcement ratio in web of right side.**

<table>
<thead>
<tr>
<th>Case</th>
<th>reinforcemnt ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.057%</td>
</tr>
<tr>
<td>2</td>
<td>0.141%</td>
</tr>
<tr>
<td>3</td>
<td>0.200%</td>
</tr>
</tbody>
</table>

![Graph](image)
connecting the loading point and the support. In Fig. 11 (b), the red colored area is where energy is absorbed. In Fig. 12 (a), the snapback process is indicated in red and the energy release and absorption during the process is shown in Fig. 12 (b), which indicates that a total of 47 J of energy is released during snapback. The snapback process is further broken down and investigated.

4.2. Diagonal tension failure process of snapback
The sudden stress drop of snapback is found to be composed of two different failure processes, i.e., a diagonal tension failure process and an anchorage failure process. Each process releases energy the relative amount of which depends on the shear span depth ratio and the extent of axial force. The existence of web reinforcement is sure to affect the amount of released energy.
This will be discussed in section 4.4.

In the case of specimen S4, the snapback failure is recognized to have initiated in the diagonal region as shown in Fig. 13 (a) and flexural deformation to have recovered with the release of energy, shown in blue in Fig. 13 (b). In other words, flexural energy was released in the central region as shown, while the failure proceeded along the diagonal area where energy was absorbed. The absorbed energy amount was smaller than the released energy by 28 J and resulted in snapback. The energy absorption and release are quantitatively shown in Figs. 13 (c) and (d), where energy is classified into concrete compression, tension, and shear. This process is called the snapback state 1 or diagonal tension failure process.

4.3. Anchorage failure process of snapback

In the next step, or snapback state 2, further failure occurred along the anchorage zone of the longitudinal bars, while energy release continued in the central flexure dominated zone. This process may be identified by examining Fig. 14. Therefore, we call this process the anchorage failure process of snapback failure. The energy release rate in the process was 19 J and the release rate in each failure process was almost the same.

Combining snapback state 1 and state 2, the total energy release during the snapback process is shown in Fig. 12 with 47 J of excessive energy release. The fact that state 1 and state 2 occur as instantaneous processes should be recognized, although it cannot be statically observed by experiment.

The existence of a post-snapback state, where a load is carried by another bearing system, is also recognized (refer to Fig. 15), which is composed of a beam section without the upper part, which is separated by diagonal cracking.

4.4. Web reinforcement effects

The web reinforcement effect on snapback was investigated performing failure calculation on beams with web reinforcement. Three beams with web reinforcement were examined as to whether snapback occurs in these cases. All three beams had the same dimensions as specimen S4 except that they have web reinforcement in the right side span. The web reinforcement ratios are shown in Table 1. The load deflection curves for these beams are compared with specimen S4 in Fig. 16. The less amount of load drop in the snapback after the peak load and the increase of the peak load deflections were observed. The energy absorption area was quite similar to specimen S4, except that energy absorption increase was very substantial in the diagonal zone between the loading point and the support point in the post snapback state.

5. Application to shear failing column analysis

The shear capacity of a column that is subjected to axial force or shear capacity of a beam with horizontal force is still needs to be clarified. There exist many experimental results for which post peak data were not obtained and for which analysis also failed to obtain the post-peak behavior.
5.1. Analysis of Y. Uchida’s specimen subjected to horizontal force

First, the same beam as the one used in Uchida’s experiment was analyzed for vertical loading while subjected to uniform horizontal loads of magnitudes 2.45 MPa and 4.90 MPa to the section, and an investigation as to what occurs in the post-peak region was performed. The axial load was applied first and the vertical load was applied subsequently to the same beam as the one used in Y. Uchida’s experiment.

The load deflection relations are shown in Fig. 18. Snapback shear failure occurred in every case with a substantial increase in shear capacity in the case of 2.45MPa axial force and a slight increase thereafter. The relative magnitude of the capacity increase is shown in Fig. 17, where it is compared with JSCE Code values. The ordinate values are normalized with the shear capacity of zero axial force for both JSCE Code values and calculated values, though the experimental value is used for normalization of calculated values.

The larger effect rate on shear capacity increase in smaller axial force may related to the affecting mechanism of axial force on shear resistance.

The failure pattern of the beams with axial force is similar to the failure pattern without axial force though flatter inclination of the localizing zone is characteristic as seen in Fig. 19. In Fig. 19, energy absorption and release after the peak load are shown for the cases with zero and 4.9MPa axial force, to make the variations in the energy absorbing area affected by the axial force more explicit.

One may be able to identify the kinking at the middle of the energy absorbing area or strut of red color in the concrete of the beam with 4.90MPa axial force and flat-
tending shape of the strut in both sides of the kinking point. This is apparently the axial force effect, which works to prevent upward buckling of diagonal struts.

A clearer picture of the characteristics is yielded by Yamaya’s experiment that follows.

During snapback failure, the load drop is eased by the increase in axial force as shown in Fig. 20. In other words, the energy release rate decreases as the axial force increases, which indicates that the damage inflicted by the release of strain energy becomes less as the axial force increase.

Therefore, in real situations, the phenomenon of sudden failure and occurrence of self vibration occurs with less frequency as axial force increases. Although this is not described in this paper, the self vibration that would follow structural snapback failure may be mathematically described.

5.2. Analysis of Yamaya’s specimen subjected to horizontal force

A more comprehensive column shear test was performed by Yamaya, et al (Yamaya, Higai and Nakamura 2002). While the main objective of the research was to calibrate the JSCE code format for axial force effect on shear capacity, this is not the object in this paper, since our aim is to investigate the mechanism of axial force effects on...
shear capacity through post-peak snapback analysis. Firstly, we examined whether Yamaya’s experiment showed good correlation with analytical results up to the maximum load point, which would ensure that the analysis of post peak snapback, which had not been obtained in the experiments, was reasonable. We selected four specimen out of 64 that had the largest shear span by depth ratio of 5.31 to observe how typical snapback is affected by axial force. The dimensions of the section and concrete properties are shown in Fig. 21 and the LECOM parameters that are used in the calculations are listed in Table 2.

The specimen used in Yamaya’s experiment are Beam No. 1, No. 2, No. 3 and No. 4. You can notice good agreement between the calculated results and the experiments for each one of the experiments, as shown in Fig. 22. The energy absorbing and releasing areas in the failing beams are shown in Fig. 23, which reveals the apparent flattening of diagonal shear crack as the axial force increases, something similarly observed in Uchida’s beams. One may be able to observe that the kinking point of energy absorbing struts in the concrete shifts to the loading point as the axial force increases, and in the energy absorbing area or red colored strut, the inclination tends to become very flat and even slope up toward the end.

This is the axial force effect and may suggest the confinement effect of axial force to prevent the diagonal strut from buckling, which is originally induced by diagonal cracking. This point should be further investigated, and may indicate another macroscopic axial force effect mechanism on shear capacity.

6. Unloading curve after peak load

One may wonder if there exists another unloading curve beside the snapback curve. The calculated curve is shown compared with the snapback curve in Fig. 24. The difference between the two is that the stress loading area vanishes altogether in the case of unloading, while there is a stress loading area in the case of snapback. Therefore, the unloading branch within the snapback segment still exists and forms a hysteretic curve as shown in Fig. 25 when load reversal occurs in the snapback segment.

7. Conclusions

This paper presents comprehensive failure mode analysis for large scale concrete structures. The analysis method is mainly targeted to catch brittle failures such as snapbacks. Energy criteria are introduced to trace failure pattern in post peak analysis.

Snapback experiments on shear failing beams carried out at Gifu University are examined and the two failure stages in the snapback process are found to consist in total snapback processes; i.e., a process where diagonal tension failure occurs in a beam between the loading point and the support and another process where the anchorage zone of longitudinal reinforcement fails. The relative energy release amount of the two failure processes is affected by the shear span and beam depth ratio as well as the axial force extent. The discussion is extended to shear failing columns and their shear capacity.

Although the paper does not include the practical design criteria of axial force effects on column shear capacity, the presented method surely assures numerical calculation enabling engineers to reasonably solve the shear capacity affected by axial forces.

References


